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March, 1959

Papers

Journal of the

IRRIGATION AND DRAINAGE DIVISION

Proceedings of the American Society of Civil Engineers

MONTHLY CONSUMPTIVE USE REQUIREMENTS FOR IRRIGATED CROPS[a]

Harry F. Blaney,[1] M. ASCE

ABSTRACT

Monthly consumptive use (evapotranspiration) data are useful in determining the disposition of precipitation and its contribution to the ground-water supply, safe yields of ground-water basins, water yields from mountain watersheds, and irrigation requirements of crops. Results of monthly determinations of evapotranspiration and transpiration for irrigated crops may be employed to plan irrigation schedules and for estimating water requirements for each crop for maximum production.

In recent years there has been considerable resurgence of research on evaporation and consumptive use of water. However there is a need for additional studies. The importance of a knowledge of water lost through evaporation and consumptive use to the efficient design and later operation of the works involved in a water-supply project has long been recognized by engineers.

It is the purpose of this paper to present data on measured monthly rates of consumptive use of water for different irrigated crops growing in Western United States and to describe a procedure for determining monthly consumptive use requirements for irrigated crops from climatological data for areas where monthly measurements of water use are not available.

INTRODUCTION

Consumptive use of water involves problems of water supply, both surface and underground, and watershed management, as well as those of the

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a. Presented at the June 1958 Convention in Portland, Oregon.

1. Irrigation Engr., Western Soil and Water Management Research Branch, Soil and Water Conservation Research Div., Agri. Research Service, U. S. Dept. of Agriculture, Los Angeles, Calif.

management and economics and multiple-purpose water projects for irrigation, power, flood control and municipal purposes. Data on monthly use of water by vegetation are essential in planning water supply projects in arid and semi-arid regions. In this paper the term "consumptive use" is considered synonymous with the term evapotranspiration. It includes all transpiration and evaporation losses from lands on which there is growth of vegetation of any kind plus evaporation from bare lands and from water surfaces.

The need for data on evapotranspiration losses by vegetation has long been recognized by administrators and engineers in regions where water rights are in dispute, or where interstate water supply and water use are not in balance. Water consumed by water-loving vegetation, such as saltcedar, cottonwoods and tules, should be given careful consideration when making an inventory of the available water supply of a river basin.[1,2,3] These uneconomic plants are spreading in the West, especially along river channels and in areas of high water table.

Consumptive use is the best index of irrigation requirements. Irrigation requirement is the amount of water, exclusive of precipitation, that is needed for the production of crops. It includes plant transpiration, evaporation, deep percolation, and other economically unavoidable wastes.

A large part of the irrigation water applied to farm land is consumed by evaporation and transpiration. In field measurements it is hard to separate evaporation from transpiration and the two processes are usually considered as one and called evapotranspiration or consumptive use.

Actual measurements of consumptive use under each of the physical and climatic conditions of any large area are expensive and time consuming. The results of research and measurements of the consumptive use of water, along with meteorological observations, provide basic data required for computing water requirements for irrigated lands where few or no data, except climatological, are available.

Early Studies in United States

Research studies have been made on evaporation from soil and evapotranspiration by federal, state, and other agencies at various times since 1900. One of the first studies of water use by irrigated crops was made in Southern California in 1903 by the Irrigation Investigations Section of the Office of Experiment Stations, United States Department of Agriculture. At various times since that date this agency, now known as the Soil and Water Conservation Research Division, has studied and measured consumptive use by different agricultural crops in various sections of the West, in cooperation with state agricultural experiment stations, state engineers, and other agencies. Evaporation, temperature, humidity, precipitation, and wind movement were usually recorded at the same time. Thus, data are available for correlating consumptive use measurements with evaporation, temperature, and other climatological observations.

Both evaporation from water surfaces and consumptive use by vegetative growth respond freely to changes in temperature, humidity, and wind movement. In connection with water supply studies of river basins, the author has used, for many years, evaporation measurements from the Weather Bureau type pan to estimate monthly consumptive use of water by natural vegetation and irrigated crops. For example, in the Pecos River Joint

Investigation,[2] Class A Weather Bureau Evaporation Stations were established at Las Vegas, Conchas Dam, Alamogordo Reservoir, Roswell, Lake McMillan, and Carsbad in New Mexico and at Red Bluff, Grandfalls, Balmorhea and Fort Stockton in Texas. Pan evaporation observations have been employed for this purpose in Arizona, California, Nevada and other Western States. Table 1 illustrates the comparison of measure use of water by orange trees and alfalfa with evaporation from a Weather Bureau type pan in San Fernando Valley, California.

Measurements of Monthly Rates of Water Use

Methods used to measure consumptive use of water by irrigated crops under field conditions were described by the author in a paper published in 1952.[4] The source of water used, whether from irrigation, precipitation, or ground water, is a factor in selecting a method. The methods usually employed in irrigation studies in Western United States to determine monthly rates of use are: Soil-moisture depletion measurements; lysimeter measurements; and empirical formulae. Soil moisture depletion studies and lysimeters have been used for many years by the author to measure rates of water consumption by irrigated crops.

In recent years, the soil-moisture depletion method usually has been employed to measure the use of water by agricultural crops growing in field plots. The change in moisture content of the soil within the root zone is measured from soil samples taken with a soil tube before and after irrigation at one-foot sections to depths of 3 to 12 feet, depending on the depth of crop roots.[4] Fig. 1 shows a set of soil sampling equipment consisting of a compressed-air unit, soil-tube jack, hand hammer and soil-sampling tube which have been employed in consumptive use, irrigation and rainfall penetration studies in Southern California. Usually, samples of soil mulch are taken separately for determining the evaporation loss after irrigation. Standard laboratory practices are used to determine the quantity of water removed from the soil by evaporation and transpiration.[4]

At various times during the past 30 years, the writer has employed this method and the results have been satisfactory.[5,6] Table 2 illustrates how the results of soil-moisture depletion studies are tabulated to show transpiration of water in one-foot sections to depth of root zone, in San Fernando Valley, California.

Results of some measured monthly transpiration by irrigated crops in Arizona and California are shown in Table 3. Consumptive use for the irrigation season can be estimated by adding about 5 inches of evaporation to the total transpiration. Table 4 gives results of some measurements of monthly consumptive use of water during the irrigation season in Western United States by various investigators.

Determining Water Use from Climatic Data

For many years irrigation engineers have used temperature data in estimating annual consumptive use of water in areas of Western United States.[1,2] In 1924 Hedke developed the effective heat method on the Rio Grande.[1] By this method consumptive use is estimated from an analysis of the heat units available to the crops of a particular valley. It assumes that there is a linear

Table 1. An Example of Measurements of Monthly Transpiration in One-foot Sections to a Depth of Five Feet by Lemon Trees Growing in Fine Sandy Loam, San Fernando Valley, Los Angeles, California 1/

Period	Number of days	Soil moisture loss, acre-inches per acre(inches) 1st foot	2nd foot	3rd foot	4th foot	5th foot	Total	Loss per 30 days
Mar 8-Mar 28	21	0.69	0.22	0.20	0.10	0.00	1.21	2.03
Mar 29-Apr 10	13	0.63	0.12	0.16	0.16	0.00	1.07	2.46
Apr 11-May 1	21	0.60	0.20	0.20	0.19	0.00	1.19	1.70
May 2 - June 5	36	1.06	0.66	0.43	0.38	0.12	2.65	2.21
June 6-June 21	16	1.04	0.48	0.39	0.18	0.09	2.18	4.08
June 22-July 20	29	0.85	0.41	0.42	0.37	0.23	2.28	2.36
July 21-Aug 23	34	1.41	1.00	0.97	0.58	0.55	4.51	3.99
Aug 24-Oct 3	41	1.75	1.38	0.58	0.58	0.50	4.79	3.51
Oct 4-Oct 31	28	0.80	0.44	0.56	0.45	0.09	2.34	2.51
Nov 1-Nov 14	14	0.74	0.22	0.03	0.18	0.01	1.18	2.53
Nov 15-Dec 6	22	0.37	0.42	0.24	0.23	0.27	1.53	2.08
Total	275	9.94	5.55	4.18	3.40	1.86	24.93	
Percent of total use		40	22	17	14	7	100	

Evaporation loss from mulch = 4.8 inches (furrow irrigation)

Consumptive use = evaporation + transpiration

= 4.8 + 24.9 = 29.7 acre-inches per acre (inches)

1/ Measurements made by Harry F. Blaney and Homer Stockwell in cooperation with Arnold Lane of the Los Angeles City Department of Water and Power.

Figure 1.--Soil Sampling Equipment

relation between the amount of water consumed and the quantity of heat available. This method and others were usually confined to estimating annual or seasonal use of water rather than monthly consumption.

In 1940 Blaney and Morin of the U. S. Department of Agriculture, in connection with the Pecos River Joint Investigation,(2,7) developed an empirical formula for computing monthly rates of evaporation and evapotranspiration from mean monthly temperature, daytime hours, and humidity records. Later, because humidity records were not readily available, Blaney and Criddle(8) simplified the Blaney and Morin formula by eliminating humidity. This method has been used by Federal and State agencies in United States and by foreign countries to compute irrigation requirements for crops growing in arid and semi-arid areas throughout the World.

Actual measurements of consumptive use under each of the various physical and climatic conditions of any large area are expensive and time consuming. The Blaney-Criddle method provides a rapid method of transferring the results of careful measurements of evapotranspiration made in several areas to other areas of similar climate. Briefly, the procedure is to correlate existing measured monthly consumptive-use data with monthly temperature, per cent of daytime hours, precipitation, growing period, or irrigation season. Coefficients so developed for different crops are used to transpose consumptive-use data for a given area to other areas for which only climatological data are available. Expressed mathematically, $U = KF =$ where

Table 2. -- Comparison of use of water by irrigated orange trees and alfalfa with evaporation from a Weather Bureau pan, San Fernando Valley, Los Angeles, California.

Month	Transpiration Use		Evapo-transpiration	Evapo-ration	Ratio of alfalfa use to pan evaporation
	Oranges 1/	Oranges 2/	Alfalfa	Pan 4/	
	Inches	Inches	Inches	Inches	
January	0.8	1.1	1.3	1.85	0.70
February	1.1	2.2	1.6	2.31	.69
March	1.4	2.3	3.1	4.44	.70
April	1.7	4.0	3.3	4.74	.70
May	2.2	4.4	6.7	7.79	.86
June	2.6	4.6	5.4 3/	8.11	.67
July	2.9	4.0	7.8	10.83	.72
August	2.7	3.4	4.2 3/	9.36	.45
September	2.6	2.8	5.6	7.95	.70
October	2.4	2.6	4.4	6.67	.66
November	1.6	2.0	3.1	4.38	.71
December	1.3	1.6	1.3	3.17	.41
Total	23.3	35.0	47.8	71.60	(Ave.) 0.66

1/ With summer cover crop.
2/ Without summer cover crop.
3/ Crop cut.
4/ Weather Bureau type pan.

U = Consumptive use (evapotranspiration) in inches for any period.

F = Sum of monthly use (f) factors for the period (sum of the products of mean monthly temperature (t) and monthly per cent of annual daytime hours (p).

K = Empirical coefficient (irrigation season or growing period).

t = Mean monthly temperature in degrees Fahrenheit.

p = Monthly per cent of daytime hours of the year.

$$f = \frac{t \times p}{100} = \text{monthly use factor.}$$

u = kf = monthly consumptive use in inches.

k = Monthly use coefficient.

Table 3. Examples of measured monthly transpiration by irrigated crops in Arizona and California determined by Soil-Moisture depletion method.

Location	Crop	Transpiration, depth in inches								Authority
		Apr.	May	June	July	Aug.	Sept.	Oct.	Total	
ARIZONA										
Phoenix	Oranges	2.5	3.3	3.9	4.4	4.2	3.8	2.6	24.70	Harris (9)
Phoenix	Grapefruit	3.3	4.1	4.9	5.6	5.7	4.9	3.4	31.9	Harris (9)
Mesa	Alfalfa a/	5.0	6.5	9.0	12.0	-	3.0	4.0	39.5	Harris (9)
Tempe	Dates	2.3	3.4	4.5	5.0	5.6	5.4	4.9	31.1	Harris (9)
Mesa	Cotton	1.1	1.6	3.5	6.8	7.0	6.0	5.0	31.0	Harris (9)
Mesa	Hegari			-	2.6	4.6	6.0	5.9	19.1	Harris (9)
Mesa	Guar			-	2.0	6.0	5.0	3.0	16.0	Harris (9)
Mesa	Soybeans			2.2	4.2	6.3	4.5	2.8	20.0	Harris (9)
Mesa	Sorghum			-	2.4	8.3	7.3	2.4	20.4	Harris (9)
CALIFORNIA										
Riverside	Oranges	2.6	3.0	3.6	4.4	4.4	3.1	2.7	23.8	Pillsbury
Riverside	Oranges	2.0	3.1	4.0	4.0	3.2	3.1	3.0	22.4	Pillsbury
Corona	Lemons	1.8	2.6	2.7	2.9	3.0	3.2	2.6	18.8	Pillsbury
Corona	Oranges	2.0	2.2	3.4	4.4	3.2	2.6	2.4	20.2	Pillsbury
Tustin	Oranges b/	1.3	1.9	2.7	3.1	2.9	2.4	1.7	16.0	Pillsbury
Anaheim	Oranges b/	1.3	2.3	3.5	3.5	2.9	2.2	1.8	17.6	Pillsbury
Azusa	Oranges b/	1.7	2.2	2.8	3.3	3.2	1.9	2.2	17.3	Blaney
Los Angeles	Lemons	1.9	2.6	2.5	3.3	3.3	3.2	2.8	19.6	Blaney
Los Angeles	Lemons	1.9	2.0	2.7	3.4	3.1	3.1	2.4	18.6	Blaney
San Bernadino	Grapefruit	2.4	3.1	3.7	4.2	4.0	3.4	2.8	23.6	Pillsbury
Escondido	Lemons b/	1.3	1.4	1.8	2.5	2.6	2.6	2.0	14.2	Blaney (5)
Los Angeles	Walnuts	3.8	4.5	5.2	5.4	4.3	2.1	1.3	26.6	Blaney (6)
Tustin	Walnuts	1.0	4.0	3.2	6.3	5.2	3.6	1.6	24.9	Beckett
Tustin	Walnuts	1.0	4.1	4.5	6.4	5.5	2.9	1.8	26.2	Beckett
Santa Ana	Oranges b,c/	1.4	1.9	2.5	2.9	2.8	2.4	1.7	15.5	Beckett
Santa Ana	Oranges b,d/	1.2	1.5	1.7	1.7	1.8	1.6	1.3	10.8	Beckett
Shafter	Cotton	0.2	1.0	3.2	7.7	8.9	5.5	3.0	29.5	Beckett
Shafter	Cotton	0.3	1.1	2.3	4.6	6.7	5.4	3.6	24.0	Beckett

a/ Evapotranspiration
b/ Coastal climate
c/ Old trees
d/ Young trees

Computations of (K) from observed data for normal water supplies and growing seasons give values for irrigated crops in arid and semi-arid areas shown in Table 5.

In order to design irrigation systems for peak use of water, there is a need for monthly coefficients as very little information has been published on this subject. For some crops like citrus, the monthly coefficients are more or less uniform throughout the irrigation season. On the other hand, the monthly coefficients for alfalfa may range from 0.65 in April for coastal areas to 1.10 in July for arid areas. Table 6 presents some values of (k) for crops having an adequate irrigation water supply, computed from formula $k = \frac{u}{f}$. Table 7 illustrates how monthly values of (k) were computed for alfalfa from measured observations at Mesa, Arizona.[9]

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Table 4. - Examples of measured monthly consumptive use (evapotranspiration) by irrigated crops during irrigation season in Western United States

Location	Crop	Consumptive use (evapotranspiration), Inches April	May	June	July	Aug.	Sept.	Oct.	Total	Authority
ARIZONA										
Mesa	Alfalfa	5.0	6.5	9.0	12.0	10.0	6.0	4.0	52.50	Harris
Mesa	Dates	6.2	7.6	8.3	9.2	8.4	7.2	5.7	52.60	Harris
CALIFORNIA										
Los Angeles	a/ Lemons	2.1	2.6	3.3	3.9	3.7	3.4	2.8	21.8	Blaney
Los Angeles	a/ Oranges	2.2	2.2	3.1	3.4	3.7	3.1	2.9	20.6	Blaney
Los Angeles	a/ Walnuts	3.8	5.0	5.9	6.1	5.0	2.8	2.0	30.6	Blaney
Los Angeles	a/ Alfalfa	3.3	6.7	5.4	7.8	4.2	5.6	4.4	37.4	Blaney
Coastal	Alfalfa	4.9	4.9	4.3	5.2	5.9	5.5	4.7	35.4	
Ontario	Peaches	1.0	3.5	6.7	8.0	6.5	2.7	1.4	29.8	Blaney
Shafter	Cotton	.5	1.0	4.0	8.5	9.7	5.8	3.2	32.7	Beckett
Firebaugh	Cotton	-	.8	1.1	7.3	7.8	3.6	2.0	22.6	Adams
Firebaugh	Cotton	-	.4	.7	8.4	9.5	3.0	2.5	24.5	Adams
Delta b/	Alfalfa	3.6	4.8	6.0	7.8	6.6	6.0	1.2	36.0	Mathew
Delta b/	Potatoes	-	1.8	4.6	6.2	3.6	1.8	-	18.0	Mathew
Delta b/	Truck	1.2	3.0	6.0	5.4	5.4	3.6	1.8	26.4	Mathew
Delta b/	Sugarbeets	1.6	3.8	6.1	7.3	6.4	2.4	-	27.6	Mathew
Delta b/	Beans	1.9	2.4	1.7	2.9	6.9	4.4	-	20.2	Mathew
Delta b/	Fruit	2.2	3.8	6.0	6.8	4.8	2.8	.8	27.2	Mathew
Delta b/	Onions	1.6	3.2	5.9	5.2	2.4	1.9	-	19.8	Mathew
Davis	Sugarbeets	-	5.2	5.7	7.1	5.8	-	-	23.8	Veihmeyer
Davis	Tomatoes	-	-	3.2	6.2	4.9	4.7	-	22.3	Veihmeyer
Davis	Alfalfa	-	6.8	7.9	8.3	7.1	4.3	-	- -	Veihmeyer
Davis	Prunes	-	5.8	6.0	7.6	6.5	5.0	-	- -	Veihmeyer
Davis	Peaches	-	5.4	6.4	7.9	7.2	5.0	-	- -	Veihmeyer
Davis	Walnuts	-	6.6	6.7	8.4	7.2	4.8	-	- -	Veihmeyer
Davis	Grapes	-	4.6	4.9	6.2	5.3	4.3	-	- -	Veihmeyer
Winters	Apricots	-	-	5.6	6.8	6.5	4.9	-	- -	Veihmeyer
NEBRASKA										
Scottsbluff	Alfalfa	1.4	4.0	7.0	7.1	6.4	3.0	-	28.9	Bowen
Scottsbluff	Beets	1.9	3.3	5.2	6.9	5.8	1.1	-	24.2	Bowen
Scottsbluff	Potatoes	-	-	-	3.4	5.8	4.4	-	-	Bowen
Scottsbluff	Oats	-	3.0	6.1	5.1	-	-	-	14.2	Bowen

a/ In San Fernando Valley, City of Los Angeles, California
b/ In Sacramento-San Joaquin Delta, California

2. **Consumptive Water Use and Requirements, by Harry F. Blaney, Paul A. Ewing, Karl V. Morin, and Wayne D. Criddle. Report of the Participating Agencies, Pecos River Joint Investigation. National Resources Planning Board. June 1942.**

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5. **Irrigation Water Equipment Studies of Citrus and Avocado Trees in San Diego County, California, 1926 and 1927, by S. H. Beckett, Harry F. Blaney and C. A. Taylor, Univ. of Calif., College of Agriculture, Agr. Expt. Sta. Bul. 489, 1930.**

Table 5.--Seasonal consumptive-use coefficients (K) for irrigated crops in Western United States

Crop	Length of growing season or period	Consumptive-use coefficient 1/ (K)
Alfalfa	Between frosts	0.80 to 0.85
Beans	3 months	.60 to .70
Corn	4 months	.75 to .85
Cotton	7 months	.65 to .75
Flax	7 to 8 months	.80
Grains, small	3 months	.75 to .85
Grain sorghums	4 to 5 months	.70
Orchard, citrus	7 months	.50 to .65
Orchard, walnuts	Between frosts	.70
Orchard, deciduous	Between frosts	.60 to .70
Pasture, grass	Between frosts	.75
Pasture, Ladino clover	Between frosts	.80 to .85
Potatoes	3½ months	.65 to .75
Rice	3 to 5 months	1.00 to 1.20
Sugar beets	6 months	.65 to .75
Tomatoes	4 months	.70
Vegetables - small	3 months	.60

1/ The lower values of (K) are for coastal areas, the higher values for areas with an arid climate.

6. Progress Report on Cooperative Research Studies on Water Utilization, San Fernando Valley, California, Irrigation Season of 1940, by Harry F. Blaney, and Homer J. Stockwell, Jr. Division of Irrigation, U. S. Dept. of Agriculture. June 1941. (Typed).

7. Evaporation and Consumptive Use of Water Empirical Formulae, by Harry F. Blaney, and Karl V. Morin, Part I, Transactions Amer. Geo. Union. 1942.

8. Determining Water Requirements in Irrigated Areas from Climatological and Irrigation Data, by Harry F. Blaney, and Wayne D. Criddle, Division of Irrigation and Water Conservation, U. S. Dept. of Agriculture, SCS TP-96, August 1950.

9. Consumptive Use and Irrigation Requirements of Crops in Arizona by Harry F. Blaney and Karl Harris. Soil Conservation Service, U.S.D.A., Dec. 1951.

Table 6. - Examples of monthly consumptive use coefficients (k) for irrigated crops based on field measurements of evapotranspiration and temperatures

Location	Crop	Monthly coefficients (k) [a]								
		Mar.	April	May	June	July	Aug.	Sept.	Oct.	Nov.
ARIZONA										
Mesa	Alfalfa	0.74	0.84	0.91	1.10	1.30	-	0.90	0.75	0.75
Mesa	Cotton	-	.30	.40	.60	.80	.80	.70	.60	-
Mesa	Soy beans	-	-	-	.35	.60	.90	.80	.50	-
Mesa	Guar	-	-	-	-	.30	.80	.90	.55	-
Phoenix	Grapefruit	.55	.65	.65	.70	.70	.75	.75	.70	.65
Phoenix	Oranges	.53	.56	.56	.58	.58	.61	.61	.61	.60
CALIFORNIA										
Coastal	Alfalfa [b]	.60	.65	.70	.80	.85	.85	.80	.70	.60
Intermediate	" [c]	.60	.70	.75	.80	.95	.95	.80	.75	.70
Interior	" [d]	.65	.70	.80	.90	1.10	1.00	.85	.80	.70
Davis	"	-	.70	.80	.90	1.10	1.00	.80	.70	-
San Joaquin Delta	Alfalfa	-	.70	.75	.85	1.00	1.00	.90	.80	-
San Fernando	"	-	.70	.80	.80	.90	.90	.90	.80	.70
Orange County	Oranges [b]	-	.40	.40	.50	.55	.50	.54	.40	.40
Los Angeles Co.	" [c]	-	.40	.40	.55	.55	.55	.55	.55	.50
" "	" Lemons [c]	-	.40	.40	.50	.50	.55	.60	.50	.40
Riverside	Oranges [d]	-	.50	.50	.55	.60	.60	.60	.60	.50
San Joaquin Delta	Beets	-	.30	.60	.85	.95	.90	.40	-	-
Davis	Beets	-	-	.80	.80	.95	.80	-	-	-
Davis	Tomatoes	-	-	-	.45	.80	.70	.80	.70	-

[a] $k = \frac{u}{f} = \frac{\text{consumptive use}}{\text{use factor}}$ = monthly coefficient. [b] Coastal climate.

[c] Intermediate climate. [d] Interior area.

Table 7. - Mean monthly temperatures, percent of daytime hours, consumptive use factor, measured consumptive use, and computed consumptive use coefficient for alfalfa, Mesa, Arizona, average 1945-1946.

Month	Mean temperatures (t)	Daytime hours (p)	Consumptive use factor (f) 2/	Consumptive use (u) 3/	Consumptive use coefficient (k) 4/
	F°	Percent		Inches	
January	48.3	7.13	3.44	1.0	0.29
February	51.9	6.93	3.60	2.0	0.55
March	56.9	8.36	4.75	3.5	0.74
April	67.3	8.79	5.92	5.0	0.84
May	73.6	9.70	7.13	6.5	0.91
June	82.1	6.67	7.92	9.0	1.14
July	89.3	9.86	8.80	12.0	1.36
September 1/ 15-30	82.6	8.34	3.44	3.0	0.87
October	68.6	7.91	5.43	4.0	0.73
November	54.7	7.04	3.85	3.0	0.78
December	50.6	6.94	3.51	2.0	0.57
Total				51.0 (U)	

1/ Rest period August 1 to September 15.
2/ $f = \frac{t \times p}{100}$ = monthly consumptive use factor . u = kf = monthly consumptive use.
3/ Measure consumptive use.
4/ Computed $k = \frac{u}{f}$ = coefficient.

The monthly percent (p) daytime hours of the year for latitudes 30 to 40 degrees north of the equator is shown in Table 8 for computing monthly consumptive use in the formula u = kf, where $f = \frac{t \times p}{100}$

Table 8 - Daytime hour percentages for each month of the year for latitudes 30 to 40 degrees north of equator. 1/

Month	Latitudes in degrees north of equator										
	30	31	32	33	34	35	36	37	38	39	40
	Percent	Percent	Percent	Percent	Percent	Percent	Percent	Percent	Percent	Percent	Percent
January	7.30	7.25	7.20	7.15	7.10	7.05	6.99	6.93	6.87	6.81	6.76
February	7.03	6.99	6.97	6.94	6.91	6.88	6.86	6.82	6.79	6.76	6.73
March	8.38	8.37	8.37	8.36	8.36	8.35	8.35	8.35	8.34	8.33	8.33
April	8.72	8.74	8.75	8.78	8.80	8.83	8.85	8.87	8.90	8.93	8.95
May	9.53	9.58	9.63	9.68	9.72	9.76	9.81	9.87	9.92	9.97	10.02
June	9.49	9.55	9.60	9.64	9.70	9.77	9.83	9.89	9.95	10.02	10.08
July	9.67	9.72	9.77	9.83	9.88	9.93	9.99	10.05	10.10	10.16	10.22
August	9.22	9.25	9.28	9.31	9.33	9.37	9.40	9.44	9.47	9.51	9.54
September	8.34	8.34	8.34	8.34	8.36	8.36	8.36	8.37	8.38	8.38	8.38
October	7.99	7.96	7.93	7.92	7.90	7.87	7.85	7.82	7.80	7.77	7.75
November	7.19	7.15	7.11	7.06	7.02	6.97	6.92	6.87	6.82	6.77	6.72
December	7.14	7.10	7.05	6.99	6.92	6.86	6.79	6.72	6.66	6.59	6.52
Total	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

1/ Computed from "Sunshine Tables", U. S. Weather Bureau Bulletin 805, 1905 ed.

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DRAINAGE OF AGRICULTURAL LANDS USING INTERCEPTOR LINES

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ABSTRACT

There is very little guidance in text books to assist the drainage engineer when he is confronted with the design of an interceptor drain line. However, some theoretical research has been done on this problem by the author and by other drainage research workers. This report gathers together some of the theories regarding the function and behavior of an interceptor drain.

INTRODUCTION

Much has been written concerning the depth and spacing of drains when the drainage problem calls for the design of a grid or herringbone system to lower a high water table in an agricultural area.(6,7) Yet, when the drainage engineer is confronted with the problem of designing an interceptor drain, there is very little technical guidance in text books to assist him.

For the past few years research work has been conducted in several high ground water areas in Soil Conservation Districts in the San Joaquin Valley of California. It has become apparent from extensive field work that in these areas the major source of seepage can be traced to extraneous sources upslope from the critical area. The remedial measures called for appear to be a series of interceptor drains to cut off this seepage and to convey the water to a point of disposal.

Because of the need for design criteria for interceptor drain systems, there has been afforded an opportunity to delve at some length into the theory of this type of drainage. This report has been written in the hope that these theories and concepts may be of use to other drainage engineers working on similar problems.

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Interceptor Drains

Interceptor drain lines are installed almost as frequently as grid systems in the irrigated areas of the West. An interceptor drain is installed to intercept lateral or horizontal flow coming from some known source upslope and thus prevent it from reaching the area to be protected. There are many instances where an interceptor line will function as the entire remedial measure.

Interceptor drains are used to entrain seepage from canals, reservoirs and other man-made sources. They are used to cut off lateral ground water movement from benches and springs or to levee-protected low lands. They have also been used to check the lateral movement of ground water flow from higher elevation irrigated areas.

The Problem

In the natural state, each problem has its own unique and complicating circumstances, but this discussion will be confined to the relatively simple general case.

The simplest problem of interception would be apparent in connection with the drainage of foreign water in an area having a uniform sloping bed of permeable soil overlying a barrier layer of relatively impermeable soil. Seepage or flow of ground water would be laterally down the slope in the permeable strata at a uniform rate. Fig. 1 is a sketch showing the situation described.

Since the permeability of the aquifer materials is considered to be uniform and the ground water flows laterally at a steady rate, we may assume that this flow can be expressed by the Darcy Law as follows:

$$Q = KIA$$

where Q = quantity of flow, K = coefficient of permeability (hydraulic conductivity) I = hydraulic gradient, and A = cross-sectional area of flow. Thus

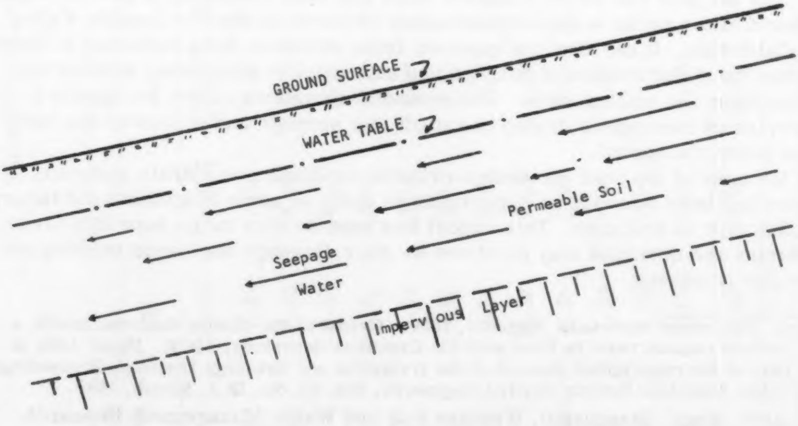

Figure 1. Lateral flow of seepage water in permeable soil overlying an impervious strata.

it is seen that the quantity of flow is a function of (a) the hydraulic conductivity, (b) the slope, and (c) the area of flow. Relative hydraulic conductivities can be measured or estimated; relative slopes can be measured; and the cross sectional area of flow can be determined. With these three factors, the total quantity of water moving downslope per unit of width is not too difficult to arrive at. But what happens to this condition when an interceptor drain is installed? The drainage engineer wants to know the answers to the following questions:

1. How deep should the interceptor drain be placed? In other words, should it be in the impervious layer? on this layer? above this layer? or halfway between the layer and the water table?
2. How far upslope will the drawdown be effective? Where does the post-installation water table become asymptotic with the undisturbed state?
3. What is the shape of the drawdown curve on the downslope side?
4. How much of the total flow will be intercepted by the device installed? In other words, what size tile or open ditch is needed?
5. Which is more effective as an interceptor device, an open drain or a closed (tile) drain?

Ideas and theoretical data have been extracted from the references cited at the end of this paper in attempting to find answers to these questions.

Depth of Drain

An interceptor drain should be placed as deep as it is practical to install. This statement might best be clarified by a few examples. It has been found by laboratory experiments and electrical analogues that the maximum amount of seepage flow would be intercepted if the drain were placed along the top of the barrier layer.[2,3,8] Thus if an open drain were to be installed so as to intersect all the soil material in the permeable strata, a maximum amount (say 95 per cent) of the foreign water would be entrained into the drain. Fig. 2 is a sketch of the situation described.

If a closed drain (tile line) interceptor were used, it appears from laboratory and field studies that the greatest amount of water would be intercepted if the tile were laid with the sand and gravel envelopes resting on the impervious layer,[4,9] as is shown in Fig. 3. If the trench of a tile line is cut into the impervious strata, there is danger that a large percentage of the water moving laterally will bridge over the tile line and continue on downslope.

Limitations may often be imposed on depth by the mechanics of installation. For example, most closed drains are limited in depth to about 7 to 8 feet. In the case of open drains, the cost of installation and the loss of agricultural land precludes their being installed at depths much below 8 feet for laterals and below 12 feet for trunk lines. While in some special instances open drains have been dug to 20 feet,[5] an interceptor open drain would seldom be constructed more than 12 feet in depth.

When it has been determined by subsurface borings that the foreign water is moving through a relatively thin confined stratum, the tile line should be installed within that stratum or the open drain should intersect it, if at all possible.[5] In these instances, the location and extent of stratum determines the depth. In actual practice, it may be impossible to fit a tile line having uniform slope into a thin confined stratum. Where these difficulties are encountered, the interceptor line should be placed within or below the thin

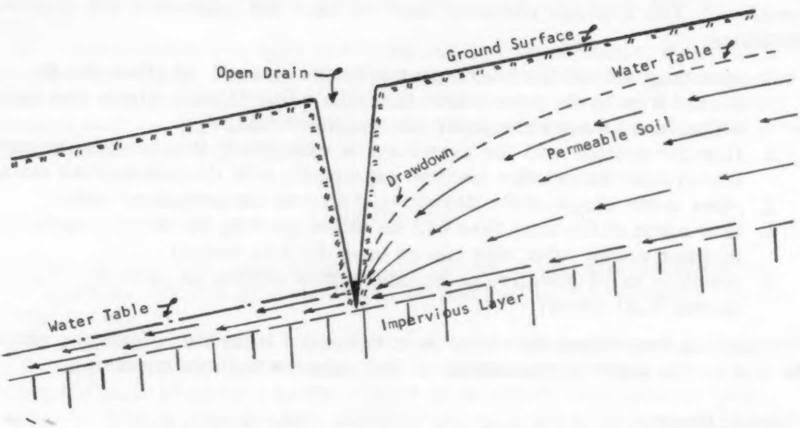

Figure 2. Interceptor drain intersecting all the permeable water bearing soil.

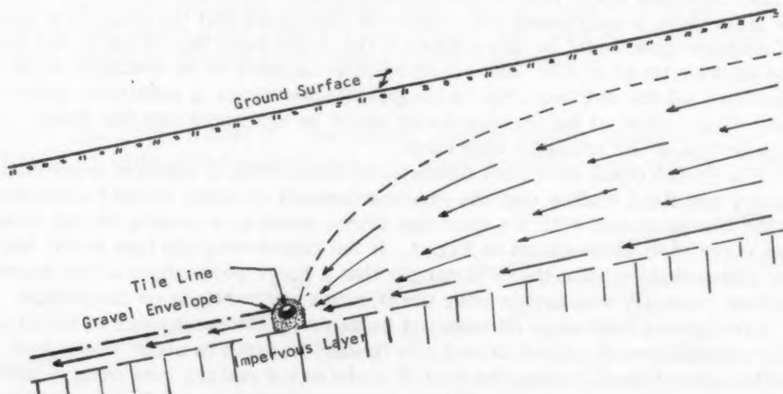

Figure 3. Interceptor tile line with gravel envelope resting on impervious layer.

confined stratum, and maximum possible continuity of flow from the stratum to the tile should be obtained by means of the sand and gravel envelope installed around the tile.

Drawdown Upslope

The drawdown or effect of the drain upslope from the site of installation presents some interesting conjectures. In an interceptor problem, the installed drain, in effect, diverts the comparatively uniform flow of an underground stream. Unlike the periodic fluctuation of water table depth from filling of the root zone by irrigation and subsequent drawdown produced by a grid drainage system, there is a comparatively uniform supply of foreign water flowing past a given point which is intercepted, in part, by the drain. For purposes of analysis, it can be reasonably assumed that this flow has reached an equilibrium and is governed by the area of flow, the nature of the soil, (hydraulic conductivity) and the hydraulic gradient (slope). If the other boundary conditions are fixed the drawdown is independent of hydraulic conductivity and quantity of flow and becomes a function of the slope. This is apparent in equations developed later.

As an example, suppose there are two soils, a coarse sand and a silt loam of the same thickness and slope underlain by barrier material and in which foreign water is flowing. Identical drains are installed in each at identical depths. What would be the effect or drawdown of the water table upslope? The drawdown would be the same for each case. Foreign water would flow more rapidly out of the coarse sand into the drain, but it would also flow into the area of the drain more rapidly. There would be more water in the drain installed in the coarse sand, but the drawdown upslope would be essentially the same as for the drain installed in the silt loam. This fact is shown by the equation derived later in this paper to describe the shape of the drawdown curve. Accordingly, if boundary conditions are the same, the same drawdown curve will develop in a sand or in a clay, and in either case the shape of the curve can be analyzed by means of the slope alone.

What then is the drawdown and at what point upslope does the drawdown merge with the undisturbed state? Laboratory experiments by Childs(3) indicate that on the uphill side of the drain the influence extends for a distance which is greater the more gradual the slope. Childs maintains that within certain limits the degree of control exerted by installation of a drain is inversely proportional to the gradient of flow. Thus if the slope were 1 foot drop in 10 feet (1 to 10), the influence upslope would be a distance of 10 feet; if the slope were 1 to 50 feet, the influence would be 50 feet; 1 to 100, 100 feet and so on.

Glover Formula

Childs' theory does not hold under varying conditions of depth of saturated flow and depth of interception. However, a very good mathematical analysis of this problem has been made by Mr. R. E. Glover of the Bureau of Reclamation.[2] Glover has developed a formula which appears to hold for practical purposes under any condition of depth of flow or interception. The derivation assumes flow parallel to the impermeable boundary and the formula does not

2. Unpublished writings of R. E. Glover, U. S. Department of Interior, Bureau of Reclamation, Denver Federal Center, Denver, Colorado.

apply in the immediate vicinity of the drain. Drainage engineers are primarily interested, however, in determining the drawdown at some distance from the drain where flow is essentially parallel, and in this area the formula holds.

In the following derivation of the Glover formula, the nomenclature has been slightly revised. Furthermore, in the final resolution there has been developed a so-called "approximation theory" which may or may not be of more use to the field technician.

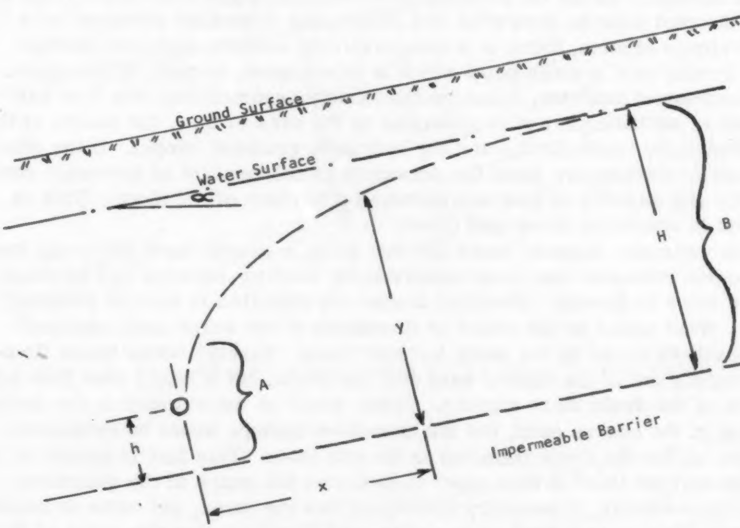

Figure 4. Site Conditions for Interceptor Drain. The above Sketch shows the site conditions of a simplified interceptor drain problem; namely, a uniformly permeable soil underlain by a relatively impermeable layer. The ground surface, underground water surface and the impermeable layer surface are all parallel.

Equation of Continuity at A

$$Q = (K)\ (y)\ \left(S + \frac{dy}{dx}\right) \qquad (1)$$

Equation of Continuity at B

$$Q = KHS \qquad (2)$$

where Q = quantity of flow per unit of width, K = coefficient of permeability, H = thickness of original saturated depth, S = slope of original water surface, h = depth from drain to impermeable barrier, and x, y, = quantities shown on figure.

Equate (1) to (2)

$$KHS = Ky\,(S + \frac{dy}{dx})$$

Cancel the K from each side

$$HS = y\,(S + \frac{dy}{dx}) = yS + y\frac{dy}{dx}$$

$$HS - yS = y\frac{dy}{dx}$$

$$S(H - y) = y\frac{dy}{dx}$$

$$S = \frac{y\frac{dy}{dx}}{(H - y)}$$

$$Sdx = \frac{(\ y\ )\ dy}{(H - y)}$$

Intergrating both sides $S \int dx = \int \frac{(\ y\ )}{H-y} dy$

(For the right hand side use the integral of the form $\int \frac{udu}{a + bu}$)

$$Sx = \left[(H-y) - H \log_e (H-y)\right] + c \qquad (3)$$

Solving for c, when $x = 0$; $y = h$, thus $c = - \left[(H-h) - H \log_e (H-h)\right]$

Substituting back into (3)

$$Sx = \left[(H-y) - H \log_e (H-y)\right] + - \left[(H-h) - H \log_e (H-h)\right]$$

$$= (H-y) - (H-h) - H \log_e \frac{(H-y)}{H-h}$$

$$= (h-y) - H \log_e (\frac{H-y}{H-h})$$

$$= (h-y) + H \left\{\log_e (H-h) - \log_e (H-y)\right\}$$

$$= (h-y) + H \log_e (\frac{H-h}{H-y})$$

$$Sx = H \log_e (\frac{H-h}{H-y}) - (y-h) \qquad (4)$$

$$x = \frac{H \log_e (\frac{H-h}{H-y}) - (y-h)}{S} \qquad (5)$$

Thus it is seen that the distance upslope (x) at which the post installation water table becomes tangential is a function not only of the slope, but also the H and h factors. In other words the influence upslope is dependent on the thickness of saturated depth and the thickness of intercepted flow. It is also seen that this formula gives infinite values of x.

In order to rationalize this approach a fixed value has been assigned to the y factor by making the assumption that the point where drawdown upslope is "significant" is that point where y factor equals 0.9H.

$$\text{Thus let } y = 0.9H$$

It is then possible to solve Eq. (5) and secure finite values for x, the point upslope where drawdown is "significant".

The Eq. (5) would become

$$x = \frac{H \log_e \frac{H-h}{0.1H} - (0.9H-h)}{S} \tag{6}$$

For given values of H, h and S we can solve Eq. (6) and secure finite values for x, the point upslope where drawdown is significant.

One additional simplification of formula (6) can be made. It is apparent that when 50 per cent or more of the flow is intercepted ($h < 0.5H$) there is little change in the value of the quantity $H \log_e \frac{H-h}{0.1H} - (0.9H-h)$. Furthermore the values of this quantity where 50 per cent or more of the flow is intercepted approaches 4/3 H. (It should be emphasized that this only holds for formula (6) based on a Y factor equal to 0.9H). Thus we can rewrite Eq. (6) into a simplified "approximation" form as follows:

$$x = \frac{4/3H}{S} \tag{7}$$

Admittedly this is an approximation, but being recognized as such it lends itself to an easy solution to the problem of estimating upslope drawdown potential of an interceptor drain.

As an example, assume that the depth of saturated flow (H) is say 9 feet and the slope is 2 feet per hundred or 0.02, and the drain device is installed so as to intercept over 50 per cent of the saturated flow. By the approximate formula (7) there would be:

$$x = \frac{4/3H}{S} = \frac{(4/3)(9)}{0.02} = \frac{12}{.02} = 600 \text{ ft.}$$

The point of "significant drawdown" would be at about 600 feet upslope.

It should be emphasized that this formula gives only approximate values for x. For precise calculations of specific problems, the use of the Glover formula is recommended where

$$x = \frac{H \log_e \frac{H-h}{H-y} - (y-h)}{S} \tag{5}$$

Drawdown Downslope

The drawdown on the downslope side is governed by the height of the water level in the drain device. This is a broad statement which needs further clarification. Obviously, any foreign water escaping below the interceptor drain will continue on downslope. In addition, some water may escape out of

the drain device itself on the downslope side. This escape occurs to the level of the water in the drain.(3) Presupposing that the same soil extends downslope from the drain, the regimen of flow imposed by permeability and gravitational forces will produce a downslope flow line of the water table approaching a parallel of that of the undisturbed state and at a lower position beginning with the height of the water surface in the drain.(1,10)

Too often it has been assumed that in order to function properly, an interceptor drain must cut off all the lateral flow. The contention being that if water escapes past the drain, the water table or flow line will tend to "climb upwards" or at least flow horizontally, while the ground slope drops downward and therefore at some distance downslope, the water level again becomes a problem. This can only occur if there is a change in the character of the soil, or a change in the hydraulic gradient downslope.

Amount of Flow Intercepted

The quantity of flow intercepted by a drain device is dependent on the type of device used and the relative position of the device in the permeable strata.(3,10) For example, if the drain device is placed at the mid-point between the water table and the impervious layer, a little less than 50 per cent of the lateral flow will be entrained; if the device is placed three-fourths of the way down, a little less than 75 per cent of the flow will be entrained. Fig. 5 illustrates this condition. Electrical analogue tests show that when the drain is placed on the impervious layer, nearly all the foreign water is caught.

In designing an interceptor drain, therefore, the engineer should first of all compute how much water is flowing past the point of interception. If a proper investigation has been made, including borings, to determine the nature and thickness of the water bearing medium, a calculation can be made of the total flow. Then by positioning the drain device in the profile, it becomes possible to make fairly close estimates of the probable quantities of water to be drained. Roughly, the quantity varies directly with the depth of flow intercepted.

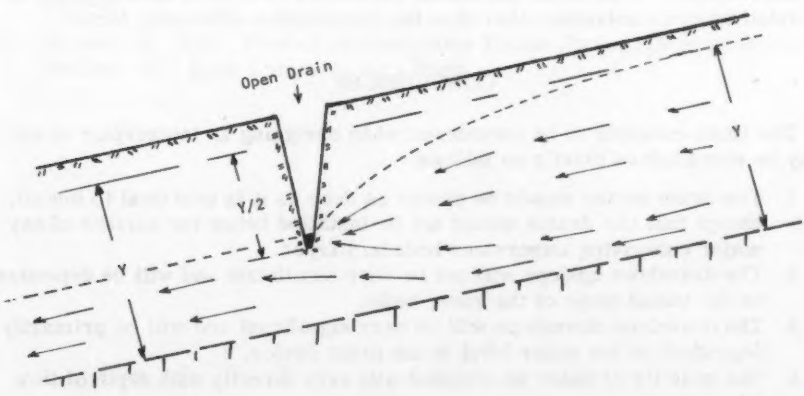

Figure 5. Open drain intercepting one-half of the depth of the water bearing stratum.

Muskat[9] proposes a formula for computing flow in an interceptor drain as follows:

$$Q_1 = (Q) \left(\frac{Y_1}{Y}\right)$$

where Q_1 = flow per lineal foot in the interceptor drain, Q = total flow per lineal foot past point of installation, Y_1 = depth of flow intercepted by drain device, Y = total depth of flow past point of installation.

Choice of Drain Type

The interceptor drain is usually installed to protect the land on the downslope side and for this purpose the choice between an open drain and a closed drain (tile line) depends on the slope of the flow. On gentle slopes and on slopes up to as steep as 1 foot drop in 30 feet, the efficiency of the two devices is about on par.[3] However, when steeper slopes are encountered, there is a bridging-over effect produced by the flow of foreign water in the capillary fringe above the phreatic water surface. This bridging effect obviously cannot occur in an open drain, but does occur in closed drains on steep slopes and decreases the percentage of water entrained. It should be noted that while the flow in the capillary fringe may not be significant where drains alternately dewater an area between irrigation cycles, it may be appreciable in the steady flow phenomenon present in an interceptor drain. Thus, when tile lines are used as interceptors on slopes steeper than 1 to 30, there will always be some bridging-over of flow. This escape over the top of the tile line ranges from 5 to 20 per cent depending on the nature of the soil and the slope.

The question might be raised as to where in agriculture would it ever be necessary to drain these excessively steep slopes? The problem would occur where an interceptor line was to be placed adjacent to and parallel with an irrigation canal, or to intercept seepage from a reservoir. Here the "slope" may be excessive. Slope in this instance would be the hydraulic gradient from water surface in the canal or reservoir to the water surface in the land adjacent. In these instances, an open drain provides the most effective type of interception. In all other instances, the choice of a drain device would be dictated by circumstances other than the comparative efficiency factor.

CONCLUSIONS

The basic concepts to be considered when designing an interceptor drain may be summarized briefly as follows:

1. The drain device should be placed as deep as it is practical to install, except that tile drains should not be installed below the surface of any major underlying impervious boundary layer.
2. The drawdown upslope will not be very significant and will be dependent on the initial slope of the water table.
3. The drawdown downslope will be very significant and will be primarily dependent on the water level in the drain device.
4. The quantity of water intercepted will vary directly with depth of flow intercepted.
5. The open drain and the tile line are equally efficient in the removal of water except on excessively steep slopes where the open drain becomes more efficient.

These concepts will be modified by the extenuating circumstances which are inherent and perhaps unique in each individual problem. The designer should keep in the forefront of his thinking the idea that in the interceptor problem he is dealing with the diversion of an underground stream, a phenomenon at considerable variance with the ordinary drainage of irrigation or rainfall patterns.

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UPSTREAM IRRIGATION IMPACT ON COLUMBIA RIVER FLOWS

Harold T. Nelson[1]

SYNOPSIS

The paper represents analyses of long-time Columbia River flows at The Dalles to demonstrate magnitude of peak flow reductions and low flow increases due to upstream irrigation. Irrigation regulations may account for up to 12% increase in low flows through irrigation of 8-1/2 million acres upstream by the year 2010.

The Columbia River, at its present level of development, and considering also its tremendous undeveloped potential, is without question the most important factor in the economy of the 259,000 square miles of river basin in the United States and Canada. The river has yielded a mean annual flow at the mouth of some 180 million acre-feet for the 50-year period 1897 to 1946, built up over a length of 1,200 miles and a fall of about half a mile. Naturally, unrelated and uncoordinated utilization of this resource started many years ago, ranging from navigation and salmon fishing in the lower reaches, to domestic, municipal, industrial, and irrigation use upstream. Correlated benefits of power, flood control, and recreation, were to come along in quick succession, and in recent years, problems of pollution and other interferences from uncoordinated operation have become apparent.

The Present Situation

At the present time, the water resource may be said to be from one-third to one-half fully utilized. There are approximately 30 million acre-feet of active storage capacity in the basin. Five million acres are irrigated of a theoretically irrigable total of some 15 million acres. The installed hydroelectric capacity in the basin is some 8,068,000 kw. of a theoretical total of

Note: Discussion open until August 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1965 is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. IR 1, March, 1959.

1. Regional Director, U. S. Bureau of Reclamation, Boise, Idaho.

37.7 million kw. The practicable upper limit of development for both irrigation and power is probably about double the existing totals.

Development experiences to date, and the size of the remaining potential, point up that river regulation will be essential for irrigation and drainage, power production, flood control, navigation, pollution control, municipal and industrial water supply, and preservation and enhancement of fish, wildlife, and recreation values. More than ever before there will be an absolute necessity for coordinated basin-wide planning, and the identification of multiple-purpose benefits, both positive and negative.

Irrigation as a Development Factor

Upstream irrigation represents the largest, as well as one of the earliest, permanent appropriations of Columbia Basin waters. In general, the economic effects of this use of waters are readily understood and accepted as a part of the every-day pattern in important valleys of the basin. However, there is very little understanding outside of the areas of application of some of the hydrologic effects of upstream irrigation, particularly, on the river regimen downstream. In recent years, downstream interests have become increasingly conscious of outright upstream depletion, with perhaps a lagging appreciation of the significance of some corollary long-range downstream flow modifications. It will be the purpose of this paper, through some analyses of the long period of recorded flows at The Dalles, Oregon, to point up reductions in peak flood flows and build-up of critical low flows that are attributable entirely to irrigation development. Brief analyses are made of the evolution of upstream storage as related to past irrigation development with estimated trends to the year 2010.

Evolution of Coordinated River Planning and Operations

It can be said that resource development projects on Columbia River have come a long way in both time and money before upstream-downstream relationships, such as those to be reported in this paper, have gained recognition as significant factors for over-all evaluation. Although 153 years have elapsed since the white man (Lewis and Clark) made the first real transit of this river, it is only within the last 25 years the coordinated operating procedures which have been accepted practice for many years on some tributaries, such as the Yakima and Upper Snake Rivers, have extended into the main stem. Up to about 1933, the bulk of water resource development projects were developed in the several states of the Columbia River drainage, as well as in Canada, by a number of private and governmental entities without benefit of more than local or at best, intrastate agreements. Where the basin may have managed for the first 125 years under more or less local auspices, the velocity of the last 25 is bringing present river development programs up against a whole host of interstate, as well as international, considerations.

Today's Problem

As the day of the single-purpose, independently developed project draws to a close, the increasingly important and competitive uses of our Columbia River water resource are forcing much more comprehensive and precise river hydrology. One of the most challenging problems confronting this river basis now and in the future is that of reaching agreement on the identification,

evaluation, and allocation of downstream benefits from upstream river regulation.

This paper, "Impact of Upstream Irrigation Development on the Downstream Regimen of Columbia River", is concerned with the hydrologic aspects of just one of the important elements in the overall problem. Of course, there are others with similar implications. Although the final solution to the problem just posed will necessarily be made at the highest levels of state and national government, the engineer and river technician will contribute mightily to the final result.

Agencies Involved

The market for river regulation data has been expanded to include a number of state, interstate, and international entities not in the picture 25 years ago. Actually, a great many of the early single-purpose water control projects, particularly those involving storage and streamflow diversions built prior to 1930, have always had measurable downstream effects and would qualify as multiple-purpose projects by today's standards. Coordinated operations on many of them are possible and have been voluntarily practiced, yielding benefits beyond the scope originally contemplated by the project sponsors.

One of the first identifications of measurable downstream effects from upstream river regulation has come about through the recognition in the water laws of the several states of the validity of appropriations of return flow for additional irrigation use. For example, a major part of the water supply to irrigate the 19,000-acre Kennewick Division of the Yakima Project, to operate the hydraulic pumping plant, and the 12,000-kilowatt power plant, is return flow by virtue of irrigation of 450,000 acres on upstream divisions of the Yakima Project.

The Federal Power Act of June 10, 1920, particularly Section 10 F, is another early example of a recognition of downstream benefits from upstream regulation. The San Francisco office of the Federal Power Commission has had a task force on Headwater Benefits Investigation—Docket No. E-6384, working on the power aspects of the over-all problem for upwards of six years.

Today the states are coming to grips with the elements of upstream-downstream relationships and other facets of water resource development through existing departments or through newly established mechanisms such as the Oregon Water Resources Board in Oregon. The Army Corps of Engineers and the Bureau of Reclamation have been exchanging data and providing a mutual evaluation of irrigation, flood control, and power benefits, for some time, particularly since the Flood Control Act of 1944. In recent years, other Federal bureaus and departments, such as the Department of Agriculture, Weather Bureau of the Department of Commerce, National Park Service, Bureaus of Sports and Commercial Fisheries, Bonneville Power Administration, and the Department of Health, Education and Welfare, have become a part of the river development program. The Water Management Subcommittee of Columbia Basin Inter-Agency Committee is an excellent example of a voluntary pooling of trained personnel for coordination of main stem operations. The Columbia Interstate Compact Commission and the International Joint Commission are two statutory commissions operating in the basin with an interest in and a requirement for the same basic data.

Existing Water-Use Pattern

As background material to the subject matter of this paper, it would be appropriate to consider the pattern of water use up to this time. Today the irrigator is the largest single user of water in the United States, and will continue to be for some years to come. On a national basis, industrial use other than for power production is in second place, and is closing the gap. For instance, today the irrigator uses about twice the industrial total. Using industrial rather than agricultural terminology for the moment, it is estimated that by the end of the next 20 years, national irrigation use will be about 170 billion gallons per day, but industrial use by that time will be 68% of the irrigation total. Of course, the relative percentage of use for irrigation in the United States portion of the Columbia River Basin is much higher and will continue to be for the foreseeable future. In the Canadian portion of the basin, irrigation use is a comparatively minor factor.

The total basin area of 259,000 square miles has yielded an average flow of approximately 180 million a.f. per year at the mouth (based on 50-year period of record 1897 to 1946), coming from a variety of hydrological environments. About 30% of the runoff at the mouth comes from the tributary area upstream from Trail, B. C., representing about 13% of the drainage area. It is significant to note, in passing, that the total Northwest water resource, including the coastal streams in the United States, yields an average flow of 205 million a.f., exceeding the combined runoff of the balance of the 17 Western states. Disregarding economic factors, there are 15 million acres of potentially irrigable land in the basin. As another oft-quoted figure, the Columbia River in various parts of its maximum fall of 6,000 feet is considered to have a theoretical hydroelectric potential of some 30 million kilowatts compared to 4-1/2 million for the Colorado; 4 million for the Missouri, and 1.1 million for the Tennessee.

Of these two principal potentials—irrigation and power generation—there is about one-third of the theoretically possible irrigation total, or about 5 million acres developed today, and on January 1, 1958 there was 21% of the theoretical ultimate, or 8.1 million kilowatts of installed hydro capacity, with another 5 million kilowatts under construction. Since the consumptive use of water for irrigation constitutes the principal depletion of water supply in the basin, it is important to have as realistic a figure as possible insofar as future irrigation development is concerned. Drawing upon work of the Water Management Subcommittee of the Columbia Basin Inter-Agency Committee, and from other sources, a realistic estimate of foreseeable irrigation development to about the year 2010 comes to about 8.7 million acres above The Dalles, or a little less than 60% of the theoretical total.

Modifications of Flow

Irrigation developments have modified downstream flows in three ways: First, there is the outright depletion due to evapotranspiration, i.e., the consumptive use of water by irrigated land; second, there is the regulatory effect of storage, some of which is held over annually and longer from periods of above-normal inflow for release in periods of critical low natural flow; and third, there is the regulatory effect of return flows throughout the year from irrigated land and associated works through surface drains and from ground water discharge into the river system.

Naturally, many factors affect the diversion requirement for a given project, such as length and type of delivery system and transit losses. Farm delivery requirements are a function of such things as length of growing season, type of irrigation practice, soil types, cropping patterns, etc. The actual farm delivery is used up in evapo-transpiration in growing the crop ("consumptive use"), which is an outright depletion of water from the system with the balance going into operational waste, surface return flow, and deep percolation losses to the ground water table. There is an annual variation in all of these factors.

Irrigation Diversions

Despite all of the above variables, it is possible to draw some fair averages based on extensive data available from operating projects. For instance, in 1957 Water District No. 36 on Upper Snake River diverted 6,400,000 a.f. to serve 993,000 acres for an average diversion of 6.5 a.f. per acre. Diversions ranged from 3.4 a.f. per acre to 12.2 a.f. per acre. Boise Valley canals diverted 1,850,000 a.f. to serve 310,111 acres for an average of 6.0 a.f. per acre. The Yakima Project diverted about 2,350,000 a.f. to serve 393,000 acres for an average of 6.0 a.f. per acre. The Columbia Basin Project diverted 1,464,000 a.f. to serve 203,000 acres for an average of 7.2 a.f. per acre. The average diversion per acre for this project will be reduced in future years as present fixed canal and lateral losses are distributed over a wider base. Irrigation diversions for the past few years reflect better than average streamflow conditions. A conservative average irrigation diversion for the Columbia Basin as a whole would be 5-1/2 a.f. per acre in an average water year.

Irrigation Depletion and Return Flow

From 3-1/2 to 4.0 a.f. per acre of the total diversion reach the farm, of which about 1-3/4 to 2.0 a.f. per acre are consumptively used in growing crops. The CBIAC studies develop a basin-wide average depletion of about 1.75 a.f. per acre irrigated. Many authorities use a rounded figure of 2.0 a.f. per acre, which, in consideration of adjacent water-using areas that are not intentionally irrigated, appears to be a good average figure. This means that for many large areas, almost two-thirds of all water diverted and one-half of that delivered to the farm actually returns to the river system as a form of naturally regulated surface and ground water return flow. Long-term readings indicate that about one-third of the total return flow comes back at a fairly uniform rate during the non-irrigation season, which coincides fairly well with the storage drawdown season of the Columbia River power system.

As a rather striking example of this latter effect, it is estimated that the Columbia Basin Project in the State of Washington at full development will return one million acre-feet per year to the Columbia River upstream from McNary Dam during the storage drawdown season. This is equivalent to an upstream storage reservoir with one million acre-feet of firm annual yield. This will be greater than the firm yield of all five reservoirs of the Yakima Project. In fact, outside of Franklin D. Roosevelt Lake and a couple of regulated natural lake storages, there are only 3 reservoirs in the basin that have a larger annual yield, namely, American Falls and Brownlee on Snake River, and Hungry Horse on the Flathead.

There are few better opportunities to measure the cumulative effects of return flow from irrigation diversions over a long period of years than in the Milner to King Hill reach of Snake River, sometimes referred to as the Thousand Springs area.

Irrigation in Snake River valley above King Hill began to get under way in 1890, with over 100,000 acres reported as irrigated. By 1920, the irrigated acreage had been increased to about 1,400,000 acres when development flattened off. Smaller increases raised the reported irrigation to 1,500,000 acres by 1940, and to 1,700,000 acres by 1955.

Inflow to Snake River, Milner to King Hill, probably averaged 5,000 c.f.s. or less prior to 1900. By 1925, return flow from irrigation had increased these inflows to over 7,300 c.f.s., and today the ground water inflows in this reach average 8,300 c.f.s. From the available data, it appears that a large proportion of the return flow developed within 10 to 15 years after first irrigation.

The effects of upstream irrigation development and the effects of storage which has been developed for supplemental supplies apparently lag much longer, and may require 20 to 40 years' time, or possibly more, to become fully reflected in the increased ground water outflows in the Thousand Springs reach of Snake River. These phenomena, by the way, will be a fruitful field for future research.

The 3,300 c.f.s. of increased flow (primarily from springs) in this reach of Snake River reflects effective ground water storage capacity having an annual yield of 2.4 million a.f. Only one surface reservoir in the whole Columbia River Basin (Lake Franklin D. Roosevelt) has a higher annual yield. It should, of course, be noted that Grand Coulee storage can be regulated as needed, whereas the ground water storage in Snake River Plain is involuntarily released at a fairly uniform rate throughout the whole year.

The Evolution of Storage

Before going into regulation through irrigation diversions as such, mention should be made of the pattern of storage development in the basin which reflects the major role of the upstream irrigator through the years.

There are now some 133 reservoirs of more than 5,000 a.f. of active capacity in the United States and Canadian portions of the basin, with a total active capacity of 28 million a.f. In addition, there are now under construction 12 other reservoirs which will bring the total close to 30-1/2 million a.f. Of the existing total, the Bureau of Reclamation has constructed or rebuilt 41 with an active capacity of 17,719,000 a.f.; the Corps of Engineers 17, containing 3,336,000 a.f.; the Bureau of Indian Affairs 15, totaling 583,000 a.f.; and private entities, state agencies, and others have built 56, capacity 5,137,000 a.f. There are 4 reservoirs in Canada with a combined capacity of 1,264,000 a.f.

A great many of these are single-purpose reservoirs, but a number, including the major storages of the Bureau of Reclamation, are operated in such a way that multiple-purpose benefits do result.

Major General Emerson C. Itschner, in a report to the Senate Committee on Interior and Insular Affairs in April 1958, stated that the total existing storage usable at-site for control of the 1894 flood crest is as follows:

"Usable at-site for control of 1894 flood
(acre-feet)

Project	River	Existing	Under Construction
Hungry Horse	Flathead	2,100,000	
Grand Coulee	Columbia	3,000,000	
Palisades	Snake	1,200,000	
Payette and Boise	Snake	650,000	
Brownlee	Snake		1,000,000
Priest Rapids	Columbia		170,000
John Day	Columbia		500,000
Subtotal		6,950,000	1,670,000
Total existing and under construction		8,620,000"	

Chart No. 1 illustrates the relationship of all storage in the basin above The Dalles to the development of irrigated land. It will be noted that prior to 1896, only unregulated natural flows were utilized. The rapid growth of irrigated land from 1896 to 1920 is also apparent, and during this period, practically all upstream storage was single-purpose irrigation impoundment.

Up to 1920, 42 reservoirs containing 4 million a.f. had been built for irrigation storage, with 8 private power reservoirs totaling 440,000 a.f. During the next 20 years (1921-1940) 38 more irrigation reservoirs were completed with a capacity of 3,580,000 a.f., and 16 reservoirs for other purposes were built, containing 3,280,000 a.f. From 1941 to 1956, 11 more irrigation reservoirs totaling 2,180,000 a.f. were built; 19 reservoirs, capacity 1,840,000 a.f., were built primarily for power; and 12 reservoirs, capacity 13,380,000 a.f. were built for multiple uses including power. The chart reflects a leveling off period from 1920 to 1950, then a sudden spurt from 1950 to the present in irrigated acres and irrigation storages, but significantly for the first time, an even greater jump in other types of upstream storage, particularly for power and flood control.

Recorded Flows at The Dalles

The flow of the Columbia River at The Dalles will be used as a convenient reference point in the charts to follow because The Dalles is downstream from practically all irrigation use except in the Willamette and has a long period of record. The charts will be based on historical and projected Columbia River flows at The Dalles, Oregon. Most of the current hydrologic analyses for Columbia River power and flood control studies by cooperating agencies are based on average flows for the 20-year period October 1928 to September 1948. Although there are records at some stations going back to 1879, the 1928-1948 period is commonly used as a measure of average long-time streamflow conditions because the record contains nearly complete data at key power sites and covers the critical 1936-1937 and 1929-1932 years as well as the high flood year of 1948. As mentioned above for Snake River, irrigation diversions began before the earliest streamflow measurements and a good part of the developed return flows are necessarily reflected in the 1928-1948 period of record. No attempt will be made here to adjust recorded flows to identify precisely before and after conditions. Nonetheless, the charts to follow, which are based on recorded and projected flows over

IRRIGATION AND STORAGE DEVELOPMENT COLUMBIA RIVER BASIN ABOVE THE DALLES

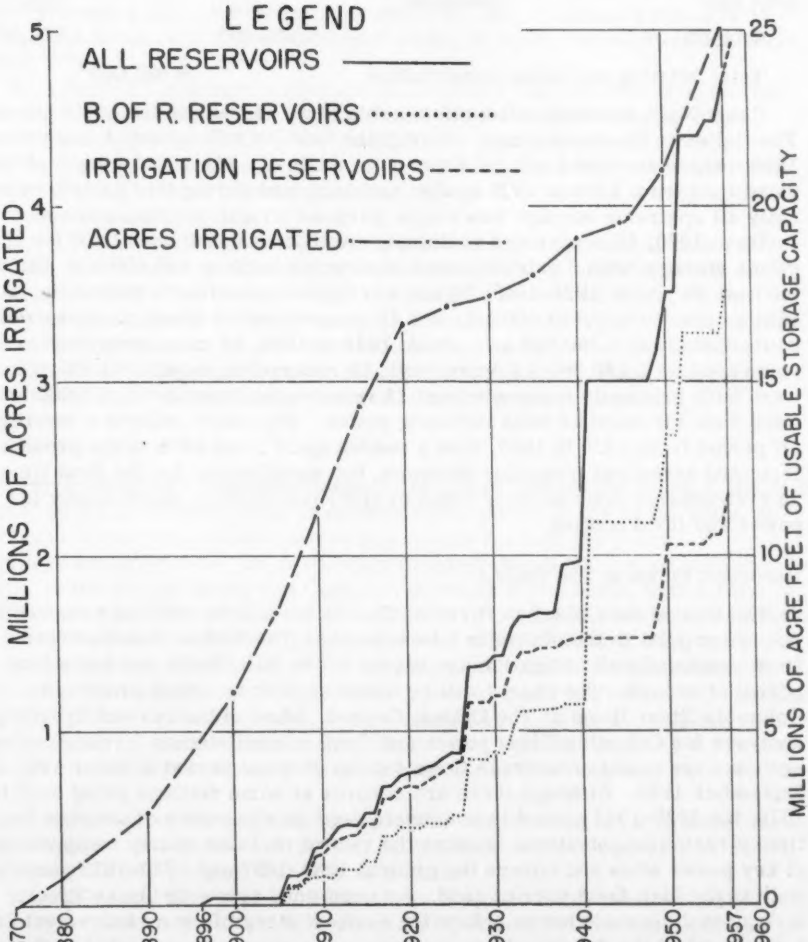


Chart 1

various periods of time, illustrate very clearly cumulative effects on downstream river regimen through upstream irrigation storage regulation and minimum flow build-up from return flow. Similarly, actual irrigated acreage for the year 1928 and projected totals for the years 1960 and 2010 will be used.

The second chart, based on the average for the 1928-1948 period, illustrates the contribution of the major tributaries to river flow at The Dalles. It will be apparent from this chart that 40% of the total flow above The Dalles originates on the Kootenai River and from the Columbia above the Kootenai; and there is very little present or potential modification of flow insofar as irrigation diversions are concerned in this area. Adding the Salmon and the Clearwater accounts for about 54% of the flow from sub-basins containing only about 3% of the existing irrigation developments, as listed in Table 1 for the year 1960.

The third chart and accompanying Table No. 1 relate average streamflows and depletions to existing irrigation developments and development expected by year 2010. The overwhelming utilization of Snake River flows for irrigation is clearly brought out with 60% of the irrigated area in the basin being confined to an area yielding less than 10% of the flow. Adding the Clark Fork-Pend Oreille, the Yakima, the Okanogan-Similkameen, the Deschutes, and miscellaneous tributaries, finds 96% of the existing irrigation development concentrated in an area yielding but 42% of the total flow at The Dalles. Nonetheless, subsequent charts will bring out the significant effects this limited placement of irrigation usage of water has had on total river flows at The Dalles.

River Flow Modifications at The Dalles

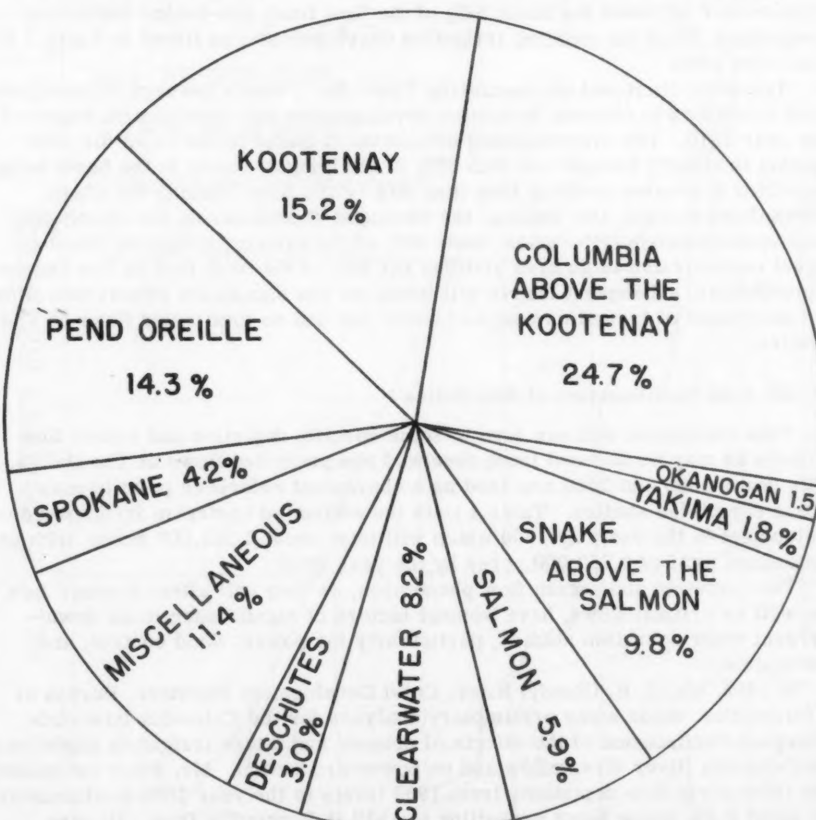
This discussion will now turn to some specific depletion and return flow effects as may be deduced from observed and projected flows at The Dalles. The years 1960 and 2010 are used as a convenient reference point to match some completed studies. Table 1 lists the estimated upstream irrigation development in the basin by 1960 which will total about 5,123,000 acres, with an estimated total of 8,752,000 acres by the year 2010.

The depletion and return flow phenomena, as they will affect average flow as well as critical flows, have become factors of significance in all downstream river operation studies, particularly for power, flood control, and navigation.

In 1955, Mr. J. R. (Randy) Riter, Chief Development Engineer, Bureau of Reclamation, made some preliminary analyses for the Columbia Interstate Compact Commission of the effects of present and future irrigation depletions on Columbia River streamflow and on power production. Mr. Riter estimated the volumetric flow depletions from 1939 levels to the year 2000 to ultimately be about 6.8% below flows prevailing in 1939 at Bonneville Dam. He also estimated that total hydroelectric energy production assuming complete development of all available head and water would be theoretically decreased about 6-1/2% for a fully regulated stream.

The CBIAC Water Management Subcommittee issued a preliminary depletion study in May of 1957. This report indicates that 1928-1948 recorded flows will be depleted about 3% at the 1960 level of development, and 8% at the 2010 level. The significant figure is that additional irrigation development over the next 50 years will result in only a 5% additional water depletion by volume.

CONTRIBUTION OF TRIBUTARIES TO FLOW OF THE COLUMBIA AT THE DALLES



AVERAGE FOR PERIOD
October 1928 - September 1948

Chart No. 2

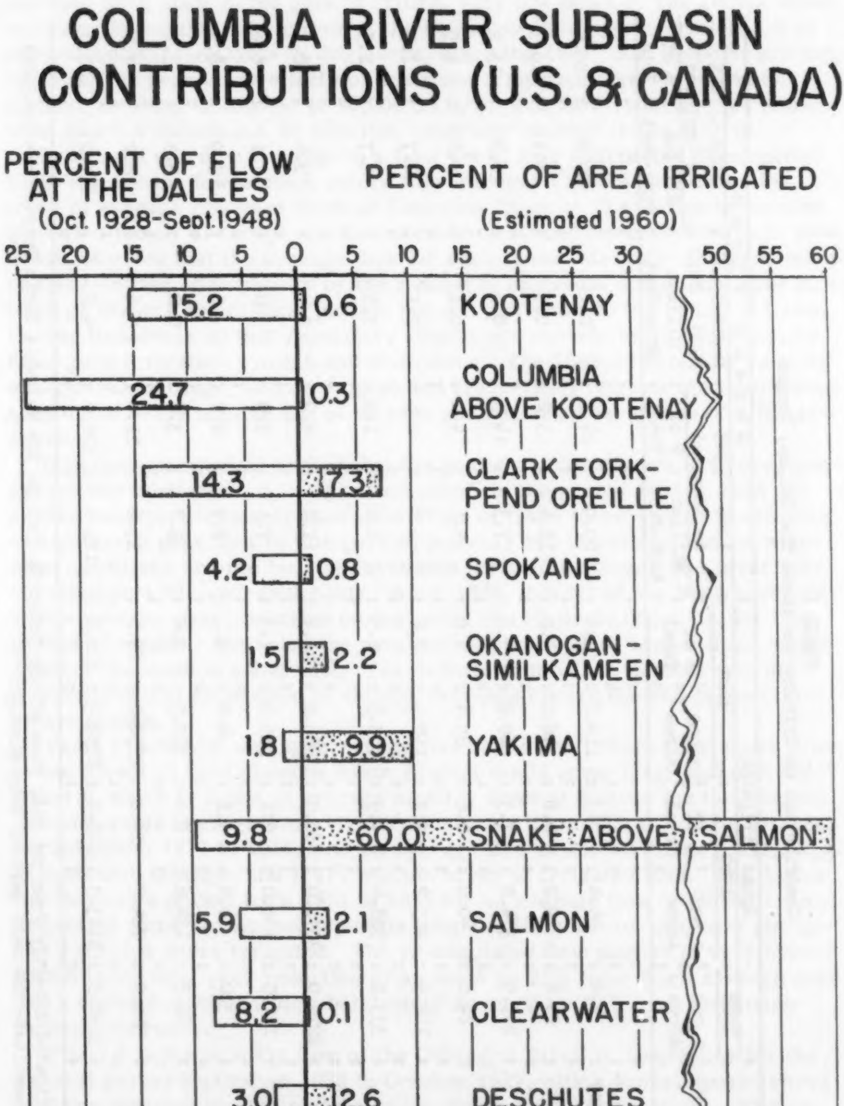

COLUMBIA RIVER SUBBASIN
CONTRIBUTIONS (U.S. & CANADA)
PERCENT OF FLOW AT THE DALLES
(Oct. 1928-Sept.1948)
PERCENT OF AREA IRRIGATED
(Estimated 1960)
25 20 15 10 5 0 5 10 15 20 25 30 50 55 60
15.2 0.6 KOOTENAY
24.7 0.3 COLUMBIA ABOVE KOOTENAY
14.3 7.3 CLARK FORK-PEND OREILLE
4.2 0.8 SPOKANE
1.5 2.2 OKANOGAN-SIMILKAMEEN
1.8 9.9 YAKIMA
9.8 60.0 SNAKE ABOVE SALMON
5.9 2.1 SALMON
8.2 0.1 CLEARWATER
3.0 2.6 DESCHUTES
11.4 14.1 MISCELLANEOUS

Chart 3

TABLE I
Contribution of Tributaries above The Dalles, Oregon

Subbasin	Areas Irrigated: 1928, 1,000 Acres	Areas Irrigated: 1960, 1,000 Acres	Areas Irrigated: 1960, % of Total	Areas Irrigated: 2010, 1,000 Acres	Areas Irrigated: 2010, % of Total	Stream Flows: 1928 - 1948 As Recorded, Million Acre-Feet	Stream Flows: 1928 - 1948 As Recorded, % of Total	Stream Flows: Depletions 1928 - 2010, Million Acre-Feet	Stream Flows: 1928 - 1948 As Depleted, Million Acre-Feet	Stream Flows: 1928 - 1948 As Depleted, % of Total
Columbia above Kootenay	14.4	14.4	0.3	119.7	1.4	29.9	24.7	0.1	29.8	26.8
Kootenay	29.2	29.2	0.6	346.1	4.0	18.4	15.2	0.3	18.1	16.3
Pend Oreille	291.9	374.0	7.3	669.2	7.6	17.3	14.3	0.5	16.8	15.2
Spokane	24.6	40.8	0.8	85.4	1.0	5.1	4.2	0.1	5.0	4.5
Okanogan	114.9	115.2	2.2	158.9	1.8	1.8	1.5	0.1	1.7	1.5
Yakima	333.1	508.4	9.9	605.5	6.9	2.2	1.8	0.5	1.7	1.5
Snake above Salmon	2278.0	3072.0	60.0	3886.0	44.4	11.8	9.8	2.4	9.4	8.4
Salmon	101.0	106.0	2.1	115.0	1.3	7.1	5.9	0	7.1	6.4
Clearwater	3.0	4.0	0.1	4.0	0.1	9.9	8.2	0	9.9	8.9
Deschutes	81.5	134.6	2.6	212.5	2.4	3.7	3.0	0.2	3.5	3.1
Miscellaneous	326.9	724.7	14.1	2550.0	29.1	13.8	11.4	5.6	8.2	7.4
Totals	3598.5	5123.3	100.0	8752.3	100.0	121.0	100.0	9.8	111.2	100.0

1/ Areas irrigated directly from the main stem of the Columbia such as the Columbia Basin Project are included in and constitute the main part of this acreage.

The above analyses yielded volumetric depletions, but no detailed analyses have been made of the gain in critical year low flows at The Dalles which increase the salable firm power of the Columbia River power system. It is understood that the Corps of Engineers, since the 1948 "308" Report, are now crediting the regulatory effects of upstream irrigation diversions and depletions as being equivalent to 40,000 c.f.s. of peak flow reduction, or something like 1.8 million a.f. of effective reservoir storage in the system.

Reference will now be made to a final set of four charts that demonstrate some significant downstream return flow effects. Chart 4 illustrates the trend of average recorded flows of Columbia River at The Dalles by months through a family of curves for four periods of record from 1888 through 1938. It will be noted that the average flow for each of the intervals is approximately equal—differing primarily by the amount of upstream depletions. The plottings go through the critical period, but end just prior to the filling of Grand Coulee Reservoir so that regulatory effects are considered to be primarily from pure irrigation storage and diversions. The 1888-1892 period reflects no upstream storage, and the last period (1934-1938) reflects the intervening addition of 8-1/2 million a.f. of storage and upstream irrigation of 3,700,000 acres.

It is to be particularly noted that the periods of flood flows have been progressively shortened. Although flood runoff in the period 1888 to 1924 occurred over progressively shorter periods of time, there was no observable diminution of peak flow in this period, possibly due to greatly altered watershed conditions and the fact that available irrigation storage was invariably operated on a fill-and-spill basis. After 1934, this trend has been reversed with maximum peak reduction on the order of 13% or 60,000 c.f.s. for that period of record. Note also the very definite build-up of winter flows with a January increase of about 46%. The definite drop-off in October was undoubtedly due to exhaustion of storage releases toward the end of the irrigation season.

From Chart 3, it was apparent that the bulk of the return flow effect prior to the advent of the Columbia Basin Project would come from Snake River. Chart 5, which is a plot of average monthly flows at Weiser for two periods of comparable average flow, vividly illustrates this. The solid line depicts periods from 1911 to 1915, with an average flow of 20,000 c.f.s., 1,561,000 a.f. of upstream storage, and 1-1/2 million acres of irrigated land. The dashed line depicts a period from 1951 to 1955 for an average flow of 19,500 c.f.s., reflecting some permanent depletion with 7 million a.f. of upstream storage and 3 million acres irrigated. The re-regulated flow pattern is very evident with a large total peak reduction of as much as 21% from April through June, and a decided minimum flow build-up of as much as 12% from December through February.

Chart 6 is the monthly flow of the Columbia River at The Dalles for the critical period September 1936 to October 1937, with a superimposed curve of flows adjusted by virtue of Columbia Basin Project operations. The reduction in June peak is the capacity of the pumping plant withdrawals less return flow, or 12,100 c.f.s. The maximum increase in flow in January would be something like 2,200 c.f.s. Of course, the area under the peak reduction of flow part of the curve exceeds that under the increased flow part of the chart by the amount of the net depletion—something like 2 million a.f. at full development.

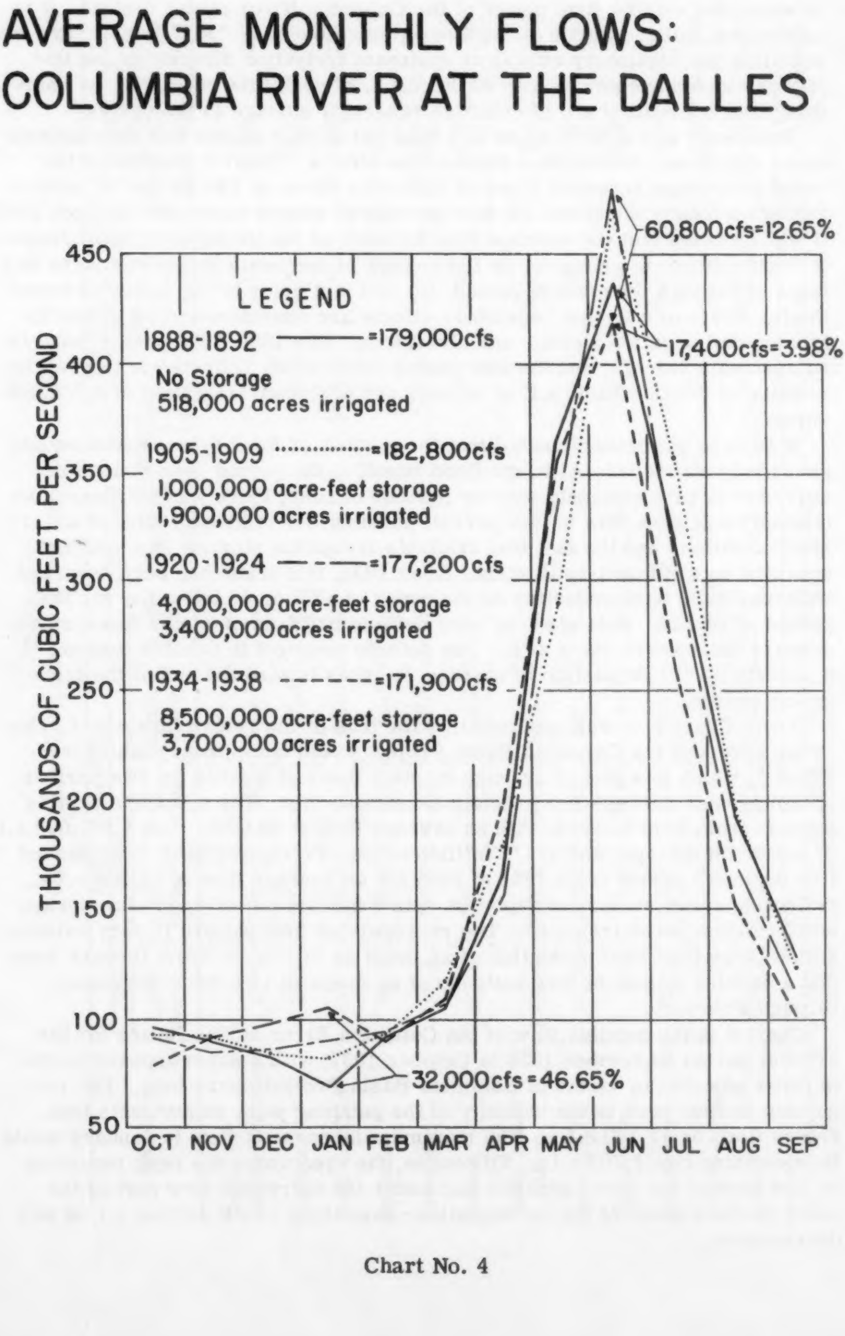

AVERAGE MONTHLY FLOWS:
COLUMBIA RIVER AT THE DALLES
LEGEND
1888-1892 =179,000cfs
No Storage
518,000 acres irrigated
1905-1909 =182,800cfs
1,000,000 acre-feet storage
1,900,000 acres irrigated
1920-1924 =177,200cfs
4,000,000 acre-feet storage
3,400,000 acres irrigated
1934-1938 =171,900cfs
8,500,000 acre-feet storage
3,700,000 acres irrigated
60,800cfs=12.65%
17,400cfs=3.98%
32,000cfs=46.65%
THOUSANDS OF CUBIC FEET PER SECOND
450
400
350
300
250
200
150
100
50
OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP

Chart No. 4

AVERAGE MONTHLY FLOWS: SNAKE RIVER AT WEISER

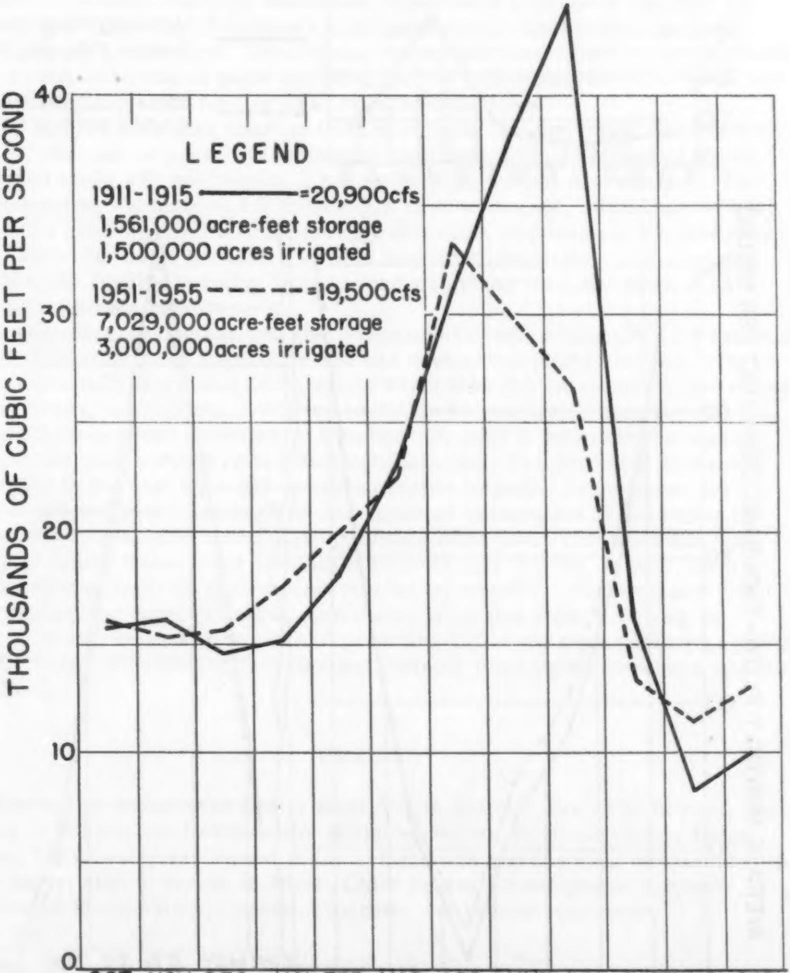


Chart No. 5

EFFECT OF COLUMBIA BASIN PROJECT ON FLOW OF COLUMBIA RIVER AT THE DALLES

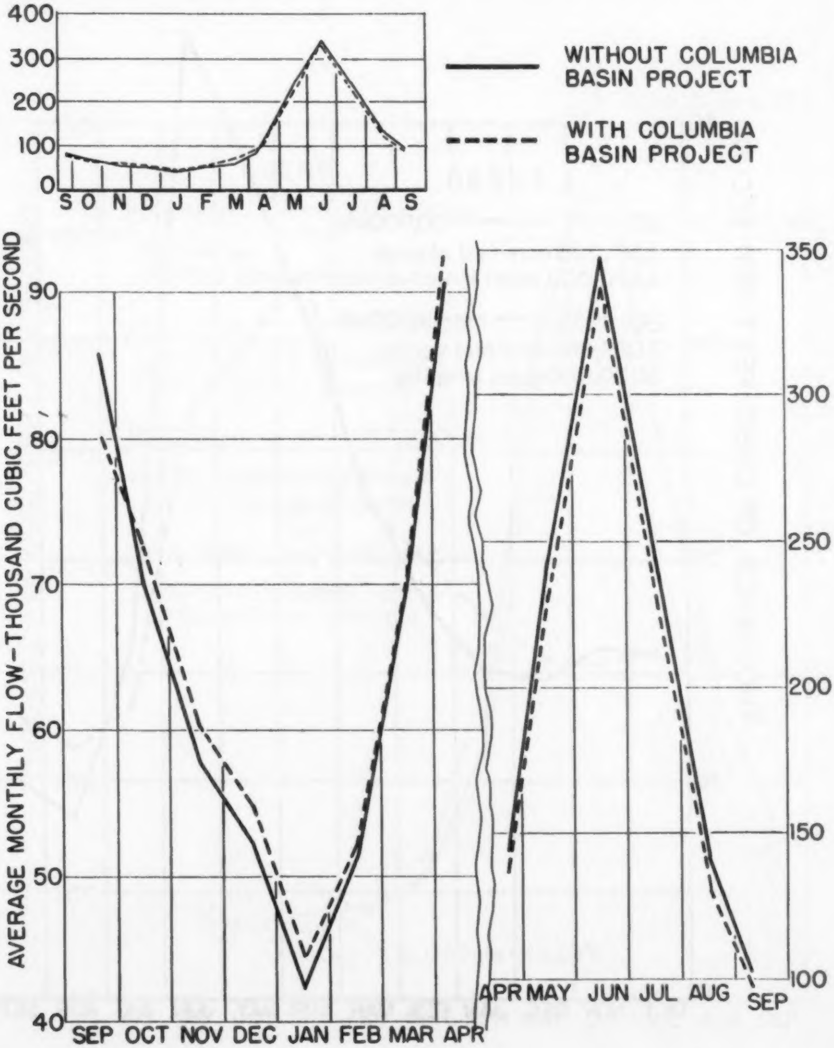


Chart No. 6

Chart 7 is a most interesting representation of the effect of all upstream irrigation on the flow of the Columbia River at The Dalles for the critical water year October 1936 to September 1937 for present-day conditions, and for conditions expected in the year 2010. The heavy black line is the average monthly flow, as recorded and, as mentioned earlier, that recorded flow does reflect at that time 7-1/4 million a.f. of upstream irrigation storage and 3.7 million irrigated acres. The heavy dotted line reflects flows adjusted to 1960 conditions with 11.9 million a.f. of upstream storage and 5.1 million acres irrigated. Finally, the right dotted line represents conditions expected to prevail in 2010 with 16.5 million a.f. of upstream irrigation storage and 8,750,000 acres irrigated. Once again, the downstream effect is unmistakable with a peak reduction of some 44,000 c.f.s., or 13% during the flood peak and an increase of 5,000 c.f.s., or 12%, on minimum flows.

The data on which the chart is built also bring out something else—namely, that for the next 50 years the estimated additional 3.6 million acres of new irrigated lands will not require a full water supply from new storage. The Subcommittee estimate of 4.6 million a.f. of new storage, which may be low, is a little over one-fourth of an average diversion requirement for this acreage. This may be so because a sizable amount of remaining irrigation opportunities will involve pumping from ground water and from the main stem without a storage requirement.

In summary, it is clear that the irrigator has been responsible for the bulk of the Columbia River regulation that has been accomplished to date. There is an extremely large part of Columbia River flow yet unregulated, particularly in Canada, susceptible to future regulation by storage. From here on out, all multiple-purpose downstream benefits will need to be evaluated more completely than heretofore to effect optimum use. The irrigator has never depended in the past upon downstream benefits to assist in the monetary return of project costs, although he has expected recognition of his rights to divert for consumptive use under applicable state law. The economic feasibility of future higher-cost irrigation projects may depend, in part, upon a recognition of indirect downstream regulatory benefits. The irrigator will not require nor expect financial assistance from this source as long as current Congressional practices of providing aid above reasonable repayment ability to otherwise feasible irrigation projects from power revenues remain in effect.

CREDITS

The writer is indebted to Mr. Boyd Austin and Mr. Harold Hafterson, engineers in the Project Development Division, Bureau of Reclamation, Boise, Idaho, for valuable assistance in the compilation and checking of data used in this paper; also to Mr. J. R. Riter, Chief Project Development Engineer, Bureau of Reclamation, Denver, Colorado, for review comments.

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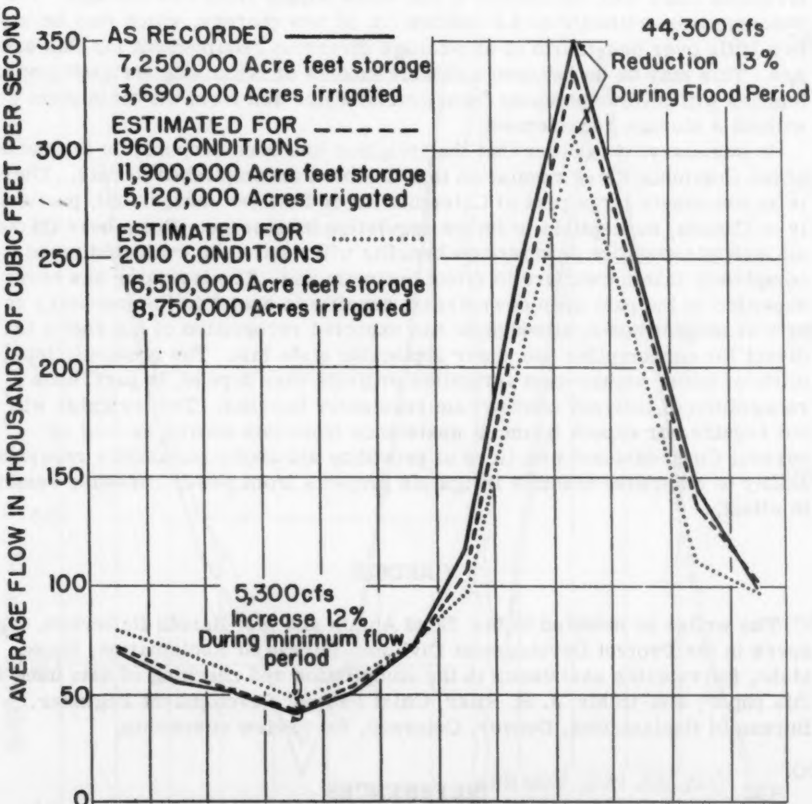
EFFECT OF IRRIGATION ON COLUMBIA RIVER AT THE DALLES

FOR WATER YEAR OCTOBER 1936-SEPTEMBER 1937



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IRRIGATION AND DRAINAGE DIVISION
Proceedings of the American Society of Civil Engineers

INVESTIGATION OF AN ARTESIAN WELL ADJACENT TO A RIVER[a]

Verne H. Scott,[1] J. M. ASCE and J. N. Luthin[2]

SYNOPSIS

A pump test of an artesian well in a problem drainage area located adjacent to a river is described. Water moved from the river through a semi-confined aquifer and then vertically upward into the problem area. Pressure conditions within the aquifer vary with changes in river stage resulting from tidal and runoff fluctuation. Calculations of aquifer transmissibilities using established well discharge relationships are made based on analysis of water level recoveries within a tidal cycle. Results indicate that pumping for drainage is not justifiable under these conditions.

INTRODUCTION

Pumping ground water for relief of drainage problems in agricultural lands is not new. The solution of several general field problems has been satisfactorily demonstrated by pumping. There are, however, many conditions where pumping has not been used and may or may not be the solution to severe drainage problems costing farmers thousands of dollars in crop and soil damage annually.

Little is known about the effectiveness of pumping on ground water contributed by a river which has changes of tidal and stage elevations. This lack of information is undoubtedly accounted for largely by a number of practical difficulties such as performing a pumping test and making sufficient and

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complete measurements of water levels, transmissibility, inflow and outflow, and evapotranspiration losses.

The purpose of this paper is to report results of a pumping test made in an area adjacent to a river where a serious drainage problem had developed. The presence of excess water often resulted in considerable, if not complete, crop damage and difficulty in performing necessary farming practices, such as plowing, discing and others. Although the area in which this study was made is somewhat unique, the conditions encountered and principles used may be common to those along other rivers or waterways.

Well Theory

Several basic concepts have been developed during recent years for flow of ground water toward wells. The basic relationship for flow through porous media is attributed to Darcy and can be written:

$$Q = K A I$$

Where: Q is the volume of water discharge per unit time; K a coefficient depending upon the medium, and I the hydraulic gradient. Dupuit applied this relationship to the case of flow toward a well within a material of homogeneous vertical cross section and assumed the hydraulic gradient to be equal to the slope of the water. The equation established by Dupuit for an artesian well is:

$$Q = 2 \pi K M \frac{h_e - h_w}{\log \frac{r_e}{r_w}}$$

Where: h_e is the depth of water at the extreme of the radius of influence, r_e, from the pumped well; h_w is the depth of water in the well, and r_w is the well radius. Q and K are terms previously defined, and M is the thickness of the aquifer.

Several investigators(1) have contributed greatly to the knowledge on groundwater flow problems and have proposed a number of solutions including special applications of nonsteady conditions, multiple wells, and boundary conditions. Contributions(2) have been made toward the application of wells to the problem of drainage of irrigated land.

The principal difficulty in applying some of the concepts developed by analytical methods is that certain ideal assumptions have been made which limit application to field problems. Rarely do field conditions conform to all of the basic assumptions of isotropic permeability and uniform thickness of the formations. Nevertheless, these analytical methods do provide a basis for the solution of many field drainage problems.

The Problem Area

Most drainage problems occur because of an accumulation of excess water moving into the area as horizontal flow or percolating downward from the soil surface.

Water can also accumulate by upward vertical movement. This condition may develop when a vertical hydraulic gradient develops causing the water to move upward creating a drainage problem at or near the soil surface. This

phenomenon reported by Israelsen and McLaughlin(3) occurred in an artesian groundwater basin in Utah. Upward hydraulic gradients of 16 feet per 100 feet were measured.

This is the type of problem occurring in many sections of the Sacramento-San Joaquin Delta of California where the investigation reported on in this paper was made.

The Sacramento-San Joaquin Delta lies in the west central part of California where the Sacramento and San Joaquin Rivers join and flow toward the ocean. It covers an area of nearly 400,000 acres of organic soils in various stages of decomposition and oxidation, together with an admixture of mineral alluvial sediments.

Throughout the area, the land surface lies approximately at sea level. At a few points elevations are five to ten feet above sea level. This accounts for the vast tulemarsh developed within the Delta. Stream beds of the large rivers and many minor waterways have divided the area to form a network of 50 interfluvial units. The banks along these channels have been built up naturally by the deposit left by flood waters and in recent years by the building of levees. Reclamation of the land between the levees has permitted extensive agricultural development of considerable importance in this area.

An increase in the facilities required to irrigate and drain this land properly has been coupled with this extensive development. Initially, simple surface ditches were dug to take care of excess surface water. Later these ditches were used to maintain the water table at a level satisfactory for some plant growth. These ditches evolved into some rather intricate and elaborate systems which are now used throughout the Delta with reasonable success. These ditch systems are not entirely satisfactory in artesian areas. Flat topography, and the presence of layers of variable thicknesses of peat soil and recent alluvial sediments of sands and gravels that readily permit lateral movement of water reduces the effectiveness of ditches.

Pumped wells offer possibilities of overcoming some of these problems which surface or tile drains cannot solve satisfactorily. Advantages may include: reduction in the number of large surface ditches, elimination of high cost of digging and laying tile drains, and removal of less land from cultivation.

Field Investigation

Some field reconnaissance, observations, and measurements were necessary in order to properly identify the problem area and determine the source and direction of ground water flow creating the drainage problem.

The surface observations and measurements included a classification of the soil, a topographic survey, and permeability measurements of the surface soil. The subsurface exploration included the installation of piezometers and the measurement of hydraulic gradients.

The surface soil close to the river was typical of the elevated alluvial ridges along the waterways of the Delta. These consist of noncalcareous mineral soils of recent alluvial accumulations and have conspicuous amounts of finely divided mica indicating granitic origin. With increasing distance from the toe of the levee, the surface slope tends to flatten out and the soil profile approaches the character of the lower lying bodies of organic soil.

Results of a topographic survey of the field under study are shown in Fig. 1. The area is bounded on the west by a ditch, levee, and the Sacramento River, and on the east by a drain ditch. Shallow 6 foot surface ditches are on both the north and south sides—750 ft. apart. The land slopes from west to east with a drop of approximately 4-1/2 feet in 1250 feet. Standing water was observed in much of the region east of the zero contour during 4 to 6 months of each year making cultivation difficult and causing crop damage.

Auger holes four to six feet deep were used to measure the soil permeability as described by Luthin.[4] Results are shown in Table 1.

These results indicate a wide variation in the permeability which can be accounted for by differences in the texture of the surface soil and the location of the bottom of the piezometer hole in relation to the underlying porous peat layer. The relationship of these measurements and the fluctuations in water table during the pumping tests will be mentioned later.

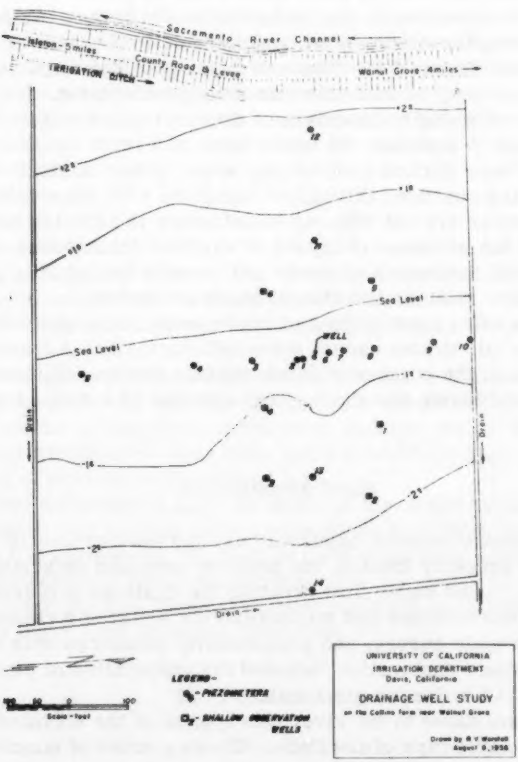


Figure 1

TABLE 1

SOIL PERMEABILITIES(1)

Location		Permeability Coefficient (K) inches per hour
West of pumped well		20.0
		13.4
	Ave.	17.7
East of pumped well		2.2
		4.9
		8.8
	Ave.	5.3

(1) Determined by auger-hole method.

A number of piezometers were jetted into the ground for the purpose of determining the type and depth of underground materials and the pressure conditions. A high pressure pump capable of developing 300 pounds per square inch was used for jetting. The material was logged during jetting, but it is difficult to identify the material accurately by this method. A typical section extending from the river to the drainage ditch on the west as obtained from the log of a number of holes is shown in Fig. 2. The mineral soil at the levee is 10 to 15 feet deep and decreased in thickness until it virtually disappeared approaching the drainage ditch on the west. Below the mineral soil was a layer of peat varying in thickness and composition. Below the peat was a layer of fine sand intermingled with streaks of clay and peat. Below the peat layer was an impermeable layer 2 to 10 feet thick of blue clay and silt. Approximately two-thirds the distance across the field this impermeable layer became discontinuous, but was detected in the log of holes jetted near the east drainage ditch. Underlying this impervious layer was a very porous layer of sand and gravel, varying in thickness from 10 to 30 feet and extending on under the river. This layer of sand and gravel offered a rather direct route for water from the river to be transmitted into the area.

The next step taken in the field was to ascertain the pressure conditions within the shallow peat layer and the underlying sand and gravel stratum. A line of piezometers was installed in approximately the middle of the field extending from the toe of the levee on the west to the drainage ditch on the east. Piezometric levels in these piezometers were observed throughout the winter of 1953-1954. Water levels in shallow piezometers extending into the organic material and in deep piezometers extending into the gravel stratum for four dates during the winter of 1953-54 are shown in Fig. 3. In general these levels showed a substantial vertical gradient and a direct relationship with river stage. Piezometric levels varied with daily tidal fluctuations which were measured in the river to have an amplitude of from 1 to 4 feet during this period of normal runoff. During flood runoff periods the river stage at this point rises 10 to 15 feet above normal levels. The phenomenon of groundwater flow related to river stage was demonstrated in a model study by Todd,[5] where a relationship between groundwater recharge and a "stage-duration" factor based on the time distribution and magnitude of groundwater flow during storm runoff periods was established.

Results of a field study by the Water Project Authority of California[6] indicated that pressure levels obtained from pipes driven to three depths showed a strong rising vertical component of groundwater during various times of the year and these variations were reported to bear some resemblance to a tidal pattern.

In Fig. 3 it should be noted that a gradient from west to east indicates lateral movement from the river. In addition, a distinct difference in piezometric pressure is noted between the shallow and the deep piezometers. Data for February 5, 1954 observations shown in Fig. 3 are plotted as a profile

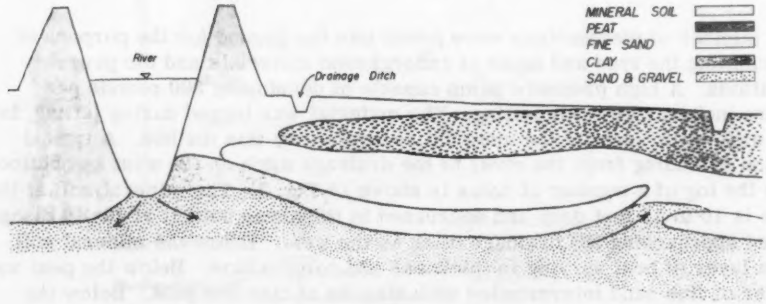
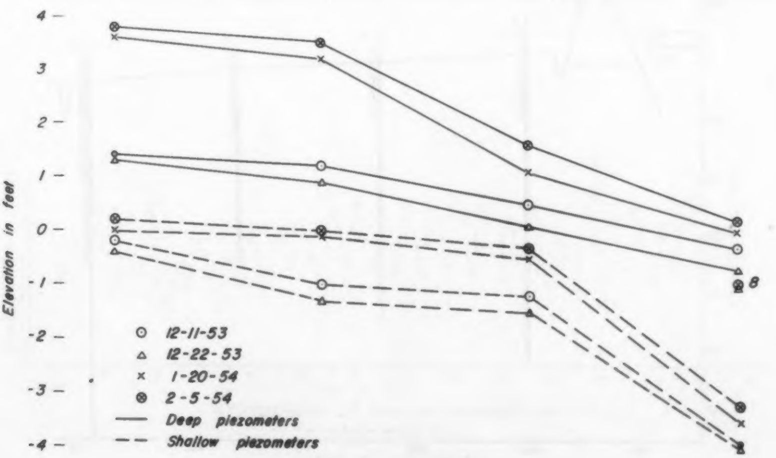


Figure 2

plot across the problem area as shown in Fig. 4. At the time of these measurements the river stage was approximately 5.0 feet above sea level. This elevation is above average for normal spring time and summer conditions. It should be noted that there is a difference of approximately one foot in elevation between the river level and the piezometric surface in piezometer 1 just inside the river levee. Further, there is a difference of 3.6 feet in elevation between the piezometric levels of piezometers 1 and 2 which represents a vertical gradient. This is a substantial gradient between the underlying sand and gravel stratum and the peat layer. This difference between the shallow and deep piezometers extends across the problem area. At the lower end of the field the difference is 3.2 feet. The piezometric level of piezometer 8 driven into the intermediate sand stratum shows an elevation in between the levels shown for the deep sand and gravel and the peat layer.

The difference in piezometric surface between the peat and the underlying sand and gravel confirmed that the source of the drainage problem in this area was water moving down and laterally from the river, and then vertically from the sand and gravel to the surface. This movement was aided by discontinuities in the confining clay stratum at a distance of 500 to 600 feet from the levee.

Two other possibilities could possibly account for changes in piezometric levels. These are: (1) variations in barometric pressure, and (2) loading on the confined acquifer by changes in tidal elevation which might possibly constitute a surload. However, the rapid response to tidal fluctuations and the relatively small amplitude of the daily tidal variations were considered to



PROFILE OF PIEZOMETERIC LEVELS

Figure 3

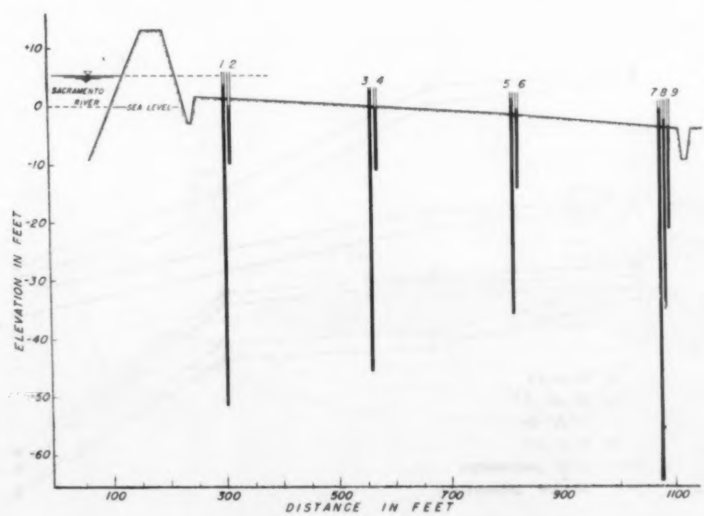
be ample evidence that variations in barometric pressure and surloading were not major factors accounting for the changes in water levels.

Pumping Tests

A preliminary pumping test from the sand and gravel aquifer was made by pumping two small wells—1-1/2 inches in diameter—jetted to a depth of 55 to 58 feet. These wells were equipped with a special well point followed by 5 feet of perforated pipe wrapped with screen. Pumping 20 to 30 gpm from these two wells for a limited period of time produced some relief in pressure in nearby piezometers. This indicated the possible success of a well of larger diameter and capacity.

An 8-inch diameter well was drilled in the position shown in Fig. 1. It was constructed by a cable tool rig to a depth of 85 feet, plugged at 77 feet, and perforated from that level up to 55 feet with vertical perforations. The log of this well was quite similar to that obtained from holes jetted nearby.

A battery of open-end piezometers, 1/2 inch in diameter, were jetted in at positions around the well shown in Fig. 1. Five piezometers were located on each side in the north-south direction and two each in the east-west direction. Each was jetted down into the underlying sand and gravel aquifer. The depth varied between 52 and 62 feet. Water level recorders were installed on Piezometers 6 and 9 and on Piezometer 3 after the third day of pumping began. The recorders were placed on 6" pipes placed over the piezometers.



PIEZOMETRIC LEVELS
ON FEB 5, 1954

Figure 4

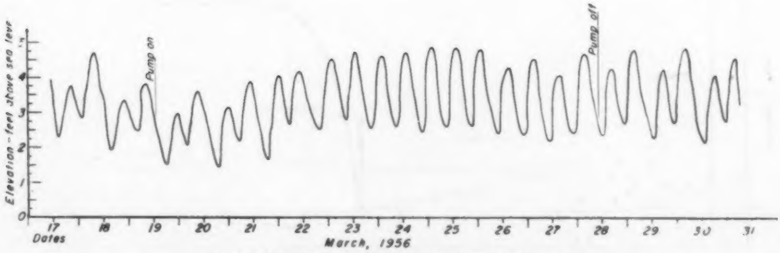
To measure changes in the shallow water table, six 6-inch diameter shallow observation wells were located around the pumped well as indicated in Fig. 1. Each of these was equipped with a continuous record water level recorder.

Water levels in the river were measured directly opposite the area by a water level recorder. Fluctuations in the river stage for the period March 17 to 31, 1956, are shown in Fig. 5. This record shows a relatively low stage in the river for the two days before the test began, followed by a gradual rise and a fairly uniform level throughout the remainder of the pump test which ended on the 28th of March. It should be pointed out that the pumping tests began at approximately the middle of a downward tidal cycle. As a result the piezometric level in all of the piezometers was moving down at the time the test was begun. Likewise, the pumping test was stopped near the end of a falling tidal cycle.

The pumping test began at 1300 March 19 and was continued until 1100 on March 28. The pump discharged only 150 to 200 gpm for the first two hours at a pumping water level of approximately 30 feet below ground level. Throughout the remainder of the test the discharge averaged 480 gpm at a pumping level of 52.0 feet. A maximum of 590 gpm was discharged for limited periods of time until the pumping water level reached the bottom of the suction pipe 62 feet below ground surface.

Pumping caused a relief in pressure in all piezometers around the well. The amount of relief varied depending on the location of the piezometer. Results obtained are shown in a series of figures:

(1) Fig. 6 shows the drawdown and recovery of Piezometer 2, 25 feet from the well. This well showed an immediate drawdown from an elevation of +2.0 feet down to elevation -2.0. The plotted points are the piezometric surface at various times throughout the test period. The dotted



HYDROGRAPH OF THE SACRAMENTO RIVER
4 miles below Walnut Grove

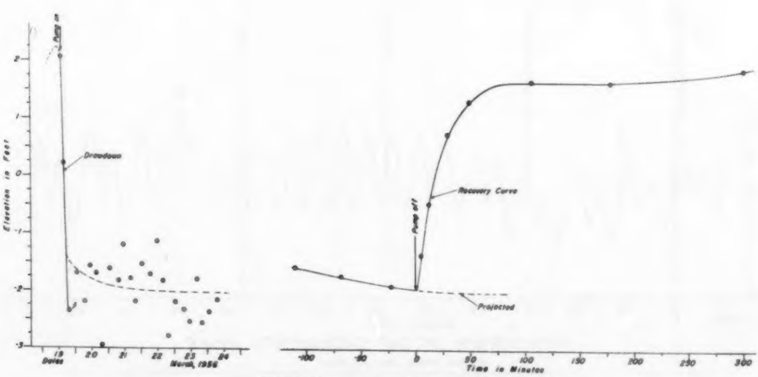
Figure 5

line represents the average pressure during this time. The right hand portion of this figure shows the measured piezometric surface prior to the time that the pump was turned off followed by a plot of the recovery curve. The piezometric level which would have occurred had the pump not been turned off is identified as "projected". An immediate response both at the start and stopping of pumping occurred in the piezometric surface.

(2) Changes in piezometric surface of Piezometer 10 are shown in Fig. 7. The initial drawdown amounted to approximately 2 feet. However, some recovery occurred followed by oscillations throughout the test period. Although pressure relief took place in this piezometer soon after pumping began, a typical recovery curve did not begin until approximately one hour later after the pump was turned off.

(3) The scattering of points during the pumping test shown in Figs. 6 and 7 suggest the possibility of two daily fluctuations in the piezometric surface. A recorder was placed on Piezometer 3—100 feet south of the pumped well—to measure these fluctuations. Results obtained are shown in Fig. 8. Although the pumping maintained an average pressure surface considerably below the initial pressure level, daily fluctuations continued to occur as a result of changes in river stage created by tidal effects.

(4) A profile of the piezometric pressure in the piezometers running north-south from the pumped well is shown in Fig. 9. The initial piezometric pressure surface represents the conditions just prior to the beginning of the pumping test. The lower levels represent the average pressure conditions throughout the pumping period. The difference in these



DRAWDOWN AND RECOVERY OF PIEZOMETER NO. 2

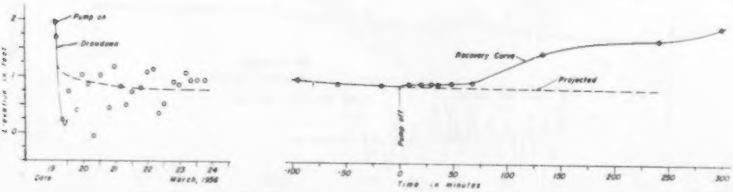
Figure 6

levels represents a typical cone of the pressure relief created by a well pumping from an artesian aquifer. It should be noted that the lower water levels of Piezometer 9 appear to be high. This may have been due to plugging of the piezometer. Although some relief in pressure was measured in all piezometers, the piezometric surface maintained by pressure in the deep sand and gravel aquifer was still above the land surface near the extremities of the problem area.

(5) The pressure relief in the sand and gravel aquifer produced some interesting effects on the shallow water table as measured by changes of water levels in shallow observation wells. Records obtained from Observation Well 3, located in the region of the poorest drainage conditions, are shown in Fig. 10. This plot shows an appreciable drop—1.5 feet—in the water table during the pumping period. This is almost the same amount as the relief in pressure recorded in the deep piezometers at about the same distance from the pumped well. It should be observed that the downward trend in the water table of this shallow well was reversed soon after the pumped well stopped. In the next 4 days the water table recovered approximately 0.5 foot above the lowest level obtained during the pumping test.

Analysis of Pumping Test Results

The rate at which a formation will transmit water is proportional to its transmissibility coefficient. This is defined as the volume of water that will flow in a unit of time under a unit hydraulic gradient through a vertical strip of a water bearing material of unit width, extending the full saturated thickness of the formation. The rate at which water is yielded from storage by a formation as the piezometric surface declines is proportional to its storage coefficient, defined as the volume of water, which a unit decline in head



DRAWDOWN AND RECOVERY OF PIEZOMETER NO. 10

Figure 7

releases from storage in a vertical prism of unit cross sectional area whose height equals the thickness of the formation. Although the conditions encountered in this field problem did not conform to the usual basic assumptions, calculations of the transmissibility were made by employing several well known relationships.

Thiem Method

Transmissibilities can be calculated using the Thiem equation as follows:

$$T = \frac{527.7\ Q\ \log_{10}(r_2/r_1)}{s_1 - s_2}$$

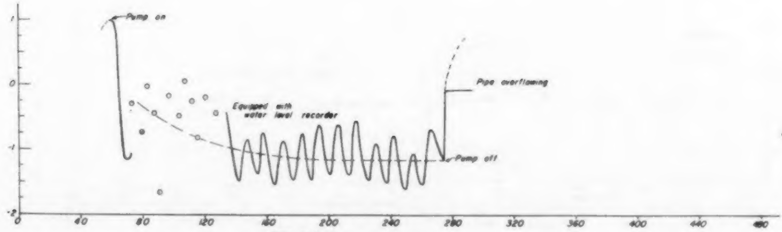
Where: T = transmissibility coefficient (defined above), r_2 and r_1 = the distance of observation wells from the pumped well, s_1 and s_2 = drawdown of water level in observation wells in feet.

Calculations of the transmissibility were based on average drawdown conditions in piezometers 50 and 100 feet from the pumped well. The drawdown in the piezometric surface was averaged for these two piezometers on either side of the pumped well. It was recognized that the values used would not represent an equilibrium condition nor could it be assumed that flow toward the well was entirely radial. Results of this calculation produced a transmissibility coefficient of 15,300 gpd per foot.

Theis Method

Theis' "non-steady equation" permits an analysis based on the rate of decline of the water level in a single piezometer within the cone of pressure relief at any time. The equation is expressed as follows:

$$s = \left(\frac{114.6\ Q}{T}\right) \int_{\frac{1.87\ r^2 S}{T\ t}}^{\infty} \left(\frac{e^{-u}}{u}\right) du$$



WATER LEVEL FLUCTUATIONS OF PIEZOMETER 3
DURING PUMPING TEST

Figure 8

Where: s = drawdown of piezometric level in feet

Q = discharge of pumped well in gallons per minute

r = distance of piezometer from pumped well in feet

T = transmissibility coefficient in gallons per day per foot

S = coefficient of storage, as a ratio or decimal

t = time well has been pumped in days

The solution of this equation has been simplified by the introduction of the term W(u) known as the "well-function of u". Rewritten the exponential integral then becomes:

$$s = \frac{114.6\ Q}{T}\ W(u)$$

The value of the integral can be found by the solution of a series where $u = \frac{1.87\ r^2 s}{T\ t}$. Values of W(u) for values of u have been presented by Wenzel.(1) A graphical method of superimposition devised by Theis greatly simplifies the use of the non-steady flow equation through the use of a "type curve". This type curve is a plot of logarithmic coordinates of the value of W(u) against values of u. By rewriting the previous equations the following are obtained:

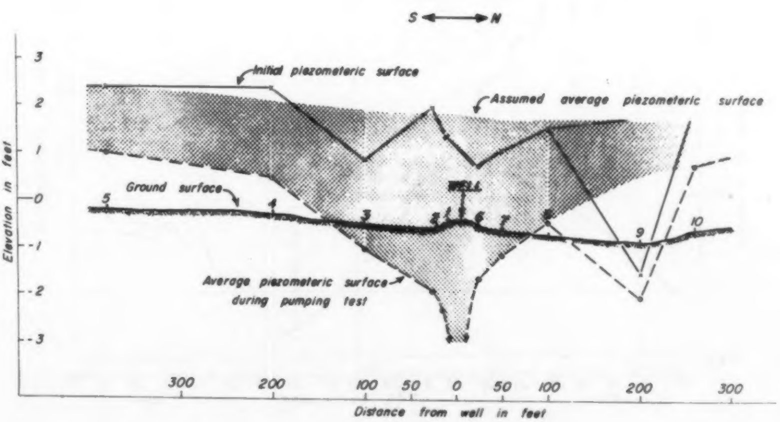


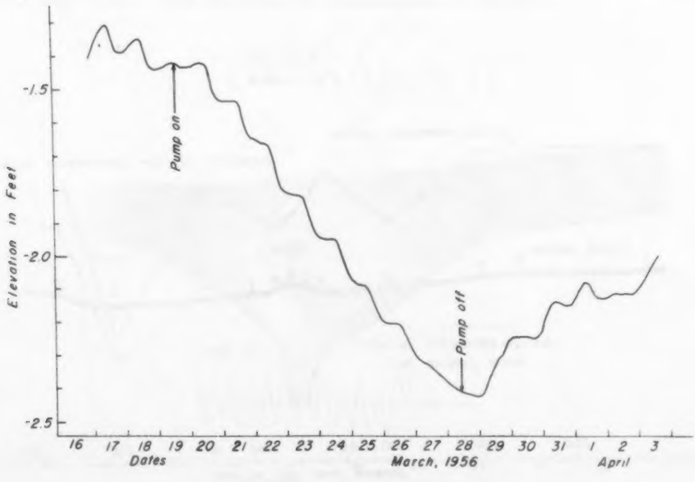
Figure 9

$$s = \frac{114.6\ Q}{T}\ W(u)$$

$$\frac{r^2}{t} = \left(\frac{T}{1.87\ S}\right) u$$

For any given pumping test the values within the bracket should be a constant providing certain assumptions are reasonably approached. Since s is related to $\frac{r^2}{t}$ as W(u) is related to u, the drawdown s occurring in any piezometer can be plotted against the values of $\frac{r^2}{t}$ on logarithmic tracing paper to the same scale as the type curve W(u) against u. Then by superposition these curves can be aligned and a matching point arbitrarily selected and values permitting calculations of T and S.

The question may be raised as to whether this method is applicable to this field case, since the non-equilibrium equation is based on the premise that all of the water discharged from a pumped well is derived from storage in an extensive reservoir and the reliability of results obtained depends largely on the length of pumping and the nature of replenishment. In this case, replenishment is related to continuity between the river and the sand and gravel aquifer, and to the level or stage of water in the river. Therefore, as a decrease in piezometric head is produced by pumping, water may not necessarily be taken from storage.



WATER TABLE FLUCTUATIONS
AT SHALLOW WELL NO. 3

Figure 10

Nevertheless, since the pressure conditions in this area were cyclic in nature depending on changes in river stage, it would appear that the non-equilibrium equations should give valid results for pumping operations not exceeding the length of one tidal cycle.

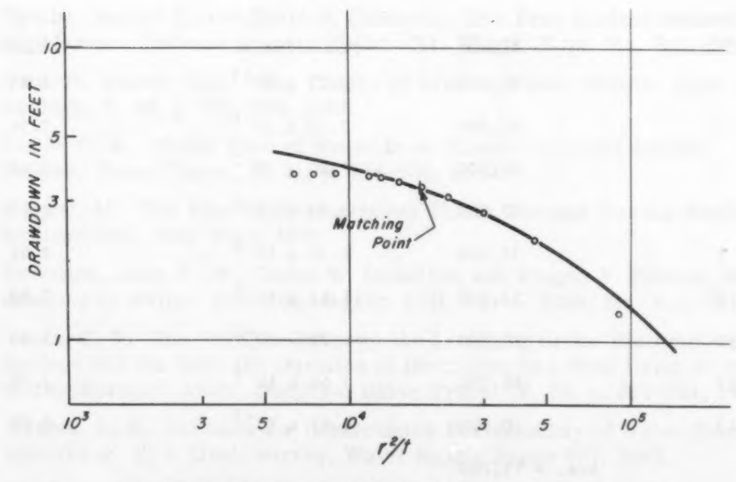
On this basis and since the river was nearing the lower end of a downward cycle at the time that the pump was turned off, it was observed that the recovery curve in the piezometers might more nearly meet the basic assumptions required for the use of the non-equilibrium method. Therefore, the estimated recovery curve was extrapolated beyond the time at which the pump was turned off for each of the piezometers and the drawdown measured from this curve to the recovery line. The recovery of Piezometer 2 is shown in Fig. 11. This is typical of the curves obtained in all the piezometers, the only difference being in the amplitude of the drawdown and the recovery curve. A summary of transmissibilities, storage coefficients and permeabilities for nine piezometers is given in Table 2.

To substantiate the results obtained by the Theis graphical method, transmissibilities were calculated using the modified equation developed by Theis, where the rate of recovery of the water level in a piezometer relatively close to the pumped well is used. The equation is:

$$T = \frac{264\ Q \log_{10} (t/t_1)}{s}$$

Where: T and Q are as defined above

t = time since pumping started



RECOVERY OF PIEZOMETRIC LEVEL

IN PIEZOMETER NO. 2

Figure 11

t_1 = time since pumping stopped

s = residual drawdown of water level in feet

The residual drawdown s is the difference between the static water level prior to the start of pumping and the water level at a given time during recovery. This definition would appear to exclude this analysis since the water level prior to the start of pumping was varying. Although the results obtained by applying this method to field data varied more than those obtained from the Theis graphical recovery method, the values of transmissibilities were close to that obtained by the other method.

Transmissibility coefficients computed by the Theis methods are reasonable considering the type of material involved. Coefficients obtained from the Thiem equilibrium relationship are considerably lower and have greater variation than those computed by the graphical method. The dynamic conditions of changing pressure conditions within the sand and gravel aquifer undoubtedly extends the application of these methods beyond the limits established in their derivation.

Further analysis of the results given in Table 2 indicates that there is not a great deal of difference in transmissibility over the problem area as determined by the Theis recovery method.

TABLE 2

TRANSMISSIBILITY, STORAGE COEFFICIENT AND PERMEABILITY OF SAND AND GRAVEL AQUIFER DETERMINED BY PUMPING TEST[1/]

Piezometer No.	T gpd/ft	S	K in/hr
1	33,200	2.27×10^{-3}	1.84
2	42,200	5.72×10^{-1}	2.34
4	56,900	7.80×10^{-3}	2.40
5	97,900	3.93×10^{-3}	5.43
7	37,300	5.32×10^{-2}	2.07
8	53,800	1.67×10^{-2}	2.99
10	36,700	1.93×10^{-2}	2.04
12	68,200	3.64×10^{-3}	3.78
13	70,500	4.87×10^{-3}	3.92
	Ave. = 59,700		

[1/] By Theis Graphical Recovery Method

CONCLUSIONS

An investigation of a drainage problem in an area adjacent to a river was made. Certain field techniques were employed to determine soil permeabilities, groundwater piezometric levels and direction of groundwater flow.

Poor drainage was attributed to upward vertical components of groundwater moving through discontinuities in a confining clay aquiclude overlying a permeable sand and gravel aquifer which has contact with the river. Fluctuations in piezometric levels reflected tidal changes in the river stage.

A pumping test demonstrated the effectiveness of producing a cone of pressure relief and in lowering the shallow unconfined water table in some areas of the field.

Recovery of water levels in deep piezometers permitted a determination of the average Transmissibility Coefficient of the sand and gravel aquifer based on the Theis non-equilibrium equation. The average value of the coefficient was 59,700 gpd/ft. Other methods of analysis did not produce consistent or reliable results.

Although the pumped well did produce some reduction in the hydraulic gradient causing flow to move toward the ground surface, it would not be economically feasible under the conditions described and with transmissibilities of the magnitude measured to drain the area by pumping.

Under different groundwater conditions a pumped well with variable discharge may offer certain advantages over other methods of drainage in areas along rivers.

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CONTENTS

DISCUSSION

SALT BALANCE IN GROUND WATER BASINS[a]

Closure by David B. Willets and Charles A. McCullough

DAVID B. WILLETS,[1] A. M. ASCE and CHARLES A. McCULLOUGH,[2] A. M. ASCE.—The writers appreciate the views and comments presented by Mr. Thomas. In his discussion Mr. Thomas has contributed to the scope of the original paper and has amplified the need for maintaining favorable salt balance in ground water basins.

The diagram entitled "Leaching Requirement as a Percentage of Applied Water" is a useful tool in planning for successful long term operation of an irrigation project when ground water is a factor in the water supply system. Mr. Thomas has also introduced the problem of drainage which is intimately related to the problem of salt balance.

The authors fully appreciate the need for, and value of, adequate drainage. The point which we wish to emphasize, however, is that advance planning for an irrigation project should include provision for an adequate supply of water to maintain salt balance. When initial plans provide for an adequate water supply to maintain favorable salt balance, the engineer has a much better basis for evaluation of possible drainage problems and more dependable data for planning drainage facilities in those instances where drainage problems may develop.

a. Proc. Paper 1359, September, 1957, by David B. Willets and Charles A. McCullough.
1. Superv. Hydr. Engr., California State Dept. of Water Resources, Los Angeles, Calif.
2. Superv. Hydr. Engr., California State Dept. of Water Resources, Sacramento, Calif.

TEAMWORK IN THE SOLUTION OF WATER PROBLEMS[a]

Closure by Harvey O. Banks

HARVEY O. BANKS,[1] M. ASCE.—In his discussion, Mr. Golze raises some very pertinent points which should be carefully considered by all who are interested in water resource development. Some degree of federal direction as a part of, or following, federal financing must be accepted; there is no reason to naively expect that the Congress would provide funds without at the same time attaching a few strings to the expenditure thereof. But federal participation of itself should not of itself necessitate federal domination. Unfortunately, the extent of direction assumed, in many cases, has been beyond that necessary to protect or promote the federal interest.

However, some degree of federal control does not mitigate the necessity for the States to exercise leadership in the various aspects of the control and development of their water resources if full benefit therefrom is to be obtained. To expect that the Federal Government will, or could, accomplish the full task of solving all water problems in all parts of the United States in time to meet the ever-expanding needs is unrealistic. The States must participate.

In the light of recent federal court decisions, however, the States can no longer expect to control the development of their water resources through the mechanism of the acquisition of water rights; the authority of the States in this regard may rest on questionable grounds, to say the least. The only effective technique of control and leadership by the States lies in active and full participation in investigations, planning and construction of water projects. All this costs money.

Mr. Golze's point concerning coordination among the many federal water resource agencies is well taken. Full coordination is not now being achieved, by any means. Mr. Golze's article is awaited with interest.

a. Proc. Paper 1497, January, 1958, by Harvey O. Banks.

1. Director of Water Resources, State Dept. of Water Resources, Sacramento, Calif.

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CAN EVAPORATION LOSSES BE REDUCED?[a]

Closure by G. Earl Harbeck, Jr.

G. EARL HARBECK, JR.,[1] M. ASCE.—Mr. Thomas' discussion is sincerely appreciated. His remarks emphasize the tremendous potential advantages that may result from storing water underground instead of in surface reservoirs. It should be remembered, however, that underground storage is not practicable at all locations. Further research on suppressing evaporation from surface reservoirs is needed, for most of the many reservoirs now in existence will continue in use indefinitely, and more will be built.

a. Proc. Paper 1499, January, 1958, by G. Earl Harbeck, Jr.
1. Hydr. Engr., U. S. Geological Survey, Denver, Colo.

RESOLVING CONFLICTING DEMANDS FOR WATER[a]

Closure by Samuel B. Morris

SAMUEL B. MORRIS,[1] M. ASCE.—Mr. Flade has confined his discussion to the Colorado River, which the author had cited as one of the interstate and international rivers used for illustrative purposes. A high degree of controversy over the water of this river has been recognized for many decades; in fact, attorneys are now engaged in preparing their findings of fact, conclusions of law and argument before filing their final briefs due by June 1, 1959, in Arizona v. California, et al. This voluminous record made before the Hon. Simon H. Rifkind, Special Master of the United States Supreme Court, required 129 court days, 22,592 pages of transcript, and 3,986 exhibits in the trial of this interstate suit filed August 13, 1952. Final decision by the Supreme Court will probably take place during the year 1960, eight years after the suit was filed.

This record affords a good example of the strife over the water of a river having inadequate flow, even when fully regulated, to meet the demands of lands and people which may be served in and outside the drainage area of 245,000 square miles, and within seven states of the United States, and Mexico. Several factors have brought this about: (1) Greatly reduced rainfall and runoff since the Colorado River Compact was entered into in 1922; (2) International agreement between the United States and Mexico requiring more than twice the quantity of water originally contemplated; (3) Lack of clarity of the Compact and other documents constituting the "Law of the River"; (4) Availability of Congressional appropriations enabling large irrigation divisions, both within and without the Colorado River drainage basin, vastly in excess of areas which would or could have been financed or their cost repaid by the areas benefited; (5) The greatest surge of population and industry to the West and Southwest that this nation has experienced.

The author has watched the growth of population in the coastal area, from Los Angeles to San Diego, during his lifetime, from 200,000 to 7,000,000 people, a growth made possible, in part, by the extensive works of the Metropolitan Water District of Southern California conveying water from the Colorado River, made available by the Hoover Dam of the United States Bureau of Reclamation. Consistent with public policy, these works are being fully paid for, including repayment of cost with interest, by the areas served. Proponents for the Colorado River Aqueduct bond issue reckoned little with the change in Federal policies of economic justification when they assured the voters that there would always be sufficient water in the Colorado River

a. Proc. Paper 1501, January, 1958, by Samuel B. Morris.

1. Cons. Engr., Los Angeles, Calif.

because irrigation could not bear the costs of full utilization of the water upstream from California diversions.

Testimony in Arizona v. California has indicated salinity values of 1.09 to 1.47 tons per acre-foot of salts. Unfortunately, there is no adequate gathering of data and continuing study by the United States to determine future salinity conditions before huge new investments are made.

The question of consumptive use of water for domestic and industrial use compared to the requirements of an equal area of irrigated land is one which must be carefully studied in each case. On the coastal plain of Southern California the density of urban population requires sewage disposal to the sea. Therefore, water requirement and consumptive use become the same. Local irrigation has been dominated by citrus culture of only 1-1/2 to 2 feet depth of applied water. While there are some urban areas that are within their limits, the total urban area, including business and industrial areas, requires about 3 feet or more. There are many inland irrigated areas which require 3 to 4 feet or more of applied water.

The relative economy of water and land use are well brought out in comparisons between the Central Arizona area and the coastal area of Southern California from Los Angeles to San Diego. Both areas began their economy on irrigated agriculture. Both are rapidly growing, but Central Arizona has just begun industrialization while Southern California has become the third industrial area in the nation in population and industry. Here are some of the comparisons:

	Central Arizona	So. California
Water applied in acre-feet	4,000,000	2,000,000
Population	700,000	7,000,000
Persons per acre-foot	0.175	3.50

In other words, an acre-foot of water on the coastal plain of Southern California supports 20 times the number of people as in Central Arizona. In terms of gross income the ratio is more than 25 times.

These figures are presented to emphasize the opportunity for industrialization in the semi-arid areas where water will do much more for the economy of the people than its major application to the growing of crops. It is not always easy to see the opportunity or bring about the transition from an agricultural to an industrial economy.

As mentioned in the author's paper, he believes it is of great importance that consideration of new areas of land to be irrigated in the semi-arid West where water supplies are limited include consideration of ultimate needs of urban population and industry. Emphasis should be given to coordinate the areas of irrigation and future urban and industrial populations.

Some time in the future history of the United States and of the world it may be necessary to utilize the maximum area for growing crops to feed man, but the time is a long way off. The leading agricultural economists have predicted the surplus crops problem in the United States in the year 2000 will not have changed much from that of the present, due to the increased yield per acre, and in spite of the great increase in population. In the meantime we can expect urban water and land requirements of an expanding population to encroach more and more upon the irrigated areas in semi-arid lands.

METHODS OF COMPUTING CONSUMPTIVE USE OF WATER[a]

Closure by Wayne D. Criddle

WAYNE D. CRIDDLE,[1] A. M. ASCE.—The application of the Blaney-Criddle "f" factor for estimating municipal water requirements by Mr. Thomas is most interesting. Although Mr. Thomas did not show individual data from which he developed his relationships, it is assumed that the correlation was reasonably good. Since the "f" factor is correlated with "water retention" in the cities, this paper lends support to the general theory that many areas under urban development in the West have consumptive water requirements similar to agriculture. With lawns, shrubs and flower beds utilizing a large proportion of the urban area, consumptive use of water should follow rather closely variations in "f" which is based on climate. Thus, the method proposed by Mr. Thomas does give a rational method of estimating water needs of cities in the West.

A discussion of this article by Mr. Blaney suggests the use of atmometers for computing consumptive use of water should be included along with the several other methods mentioned. The author is familiar with atmometers, having used them on water requirement studies, but believes there are several limitations to their use.

First, when estimates of water requirements are needed, and particularly for new projects, there are seldom atmometer data available for the area in question. Even if readings are available, knowledge on how to apply the data is still lacking. Readings made in an arid region before development are much different than those obtained at the same spot after all the land is placed under irrigation.

Second, reliability of readings is often questionable. The distance the atmometers are placed above the growing, their relationship to surrounding dry areas and the prevailing winds, and many other factors appear to greatly influence atmometer readings.

Thus, although various investigators have shown some excellent correlations for short periods, general application of the method to widely varying conditions has not yet been satisfactorily demonstrated so far as the author knows. Therefore, the method may need further study before it can be considered as one of those generally accepted and used by professional engineers and agronomists responsible for the planning and design of large irrigation projects.

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a. Proc. Paper 1507, January, 1958, by Wayne D. Criddle.

1. Utah State Engr., formerly Irrig. Engr., Western Soil and Water Management Branch Soil and Water Conservation Div., Agri. Research Service, U. S. Dept. of Agri., and Prof. of Civ. Eng., Utah State Univ., Logan, Utah.

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USBR'S LOWER-COST CANAL LINING PROGRAM[a]

Closure by R. J. Willson

R. J. WILLSON,[1] M. ASCE.—The economy and advantage of using unreinforced, monolithically cast-in-place concrete pipe in lieu of canal linings should be considered, as suggested by Mr. Hotes, particularly for small or secondary irrigation carriage and distribution systems. The miles of "two-part" type cast-in-place concrete conduit placed by the Turlock Irrigation District to serve lands in the San Joaquin Valley of California are reported to be serving very well. The installation by the Salt River Valley Water Users' Association on the Salt River Project, Arizona, of over 21 miles of 30-inch and 42-inch diameter, unreinforced, cast-in-place, monolithic pipe of the "no-joint" type, was described in the November 1957 issue of Western Construction News.[2]

Since the Western Construction News article was written, the Association has constructed additional miles of this type of pipe as a replacement for deteriorated conventional canal linings, for enclosing open ditches in urban areas and in lieu of linings in unlined reaches of other laterals. In addition to the 30-inch and 42-inch diameter pipe, a machine now has been purchased to construct 54-inch diameter conduit. The cost of constructing the smaller sizes of pipe is reported by the Association to be:

Diameter	Number of Installations	Cost Per Linear Foot	
		Range	Average
30-inch	10	$2.91 to 3.90	$3.10
42-inch	7	4.38 to 6.00	5.13

The range in costs is attributable to subgrade conditions and other preparatory factors encountered incidental to the installation of the pipe.

Approximately 2,000 linear feet of 30-inch and 1,300 linear feet of 48-inch conduit of the no-joint type also was placed experimentally in 1954 on the Orland Project in Northern California.

Experience with the cast-in-place concrete conduit in the central Arizona and northern California areas has been very satisfactory. It is believed the climate and soil conditions on these projects are almost ideal for this type of construction and that these conditions are important in considering the use of either the "two-part" or "no-joint" type of pipe. Temperate climate and cohesive soils that will stand almost vertically for trench excavation are believed essential to the economical and possibly successful construction of this

a. Proc. Paper 1589, April, 1958, by R. J. Willson.
1. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.
2. Cast-in-place pipe for irrigation, Western Construction News, November 1957.

type of conduit. In unconsolidated and noncohesive soils and under severe climatic conditions the acceptability and practicability of the pipe is still to be determined. Future planned installations in other areas will be closely followed.

A close examination of the interior of the "no-joint" pipe on the Salt River Project, the first of which was placed in 1955, has not been possible because of its being in constant use or full of water; however, there has been little evidence of leakage or cracks. An evaluation was made in 1957 of the 30-inch "no-joint" pipe placed on the Orland Project and tests to determine the leakage in a section of pipe were conducted. The indicated losses were about 0.5 per cent based upon a designed capacity of 10 cfs. The leakage was primarily from construction joint cracks at the junction of the conduit with the headworks and outlet structures. Provisions have been made for sealing these leaking joints and additional tests will be conducted as operating conditions permit.

Internal hydraulic pressure and external loading to which this pipe should be subjected also is a factor to be considered. Tests made by the Salt River Valley Water Users' Association (2) and by the Bureau of Reclamation on the Orland Project indicate the capability of the pipe to withstand considerable more stress than the internal hydraulic pressure equivalent to 8 to 10 feet of head that the installed pipe has been subjected to during normal operation to date. The "no-joint" pipe is being used on the Salt River Project in urban and rural areas. Conventional precast concrete pipe is being specified for use in locations subject to external loading and shock such as that encountered under streets and highways.

WATER—A LIMITING RESOURCE ?[a]

Discussions by Douglas R. Woodward, Alfred R. Golze and Donald McCord Baker

DOUGLAS R. WOODWARD,[1] A. M. ASCE.—The author has presented an interesting and provocative paper of concern to engineers and laymen alike. While one may differ in degree with his conclusions, he has very ably demonstrated that the future problems of supplying water to the many needs of the United States will be both serious and challenging. The author also presents convincing evidence to show that we must learn to consider water as a national resource as well as a local or regional resource.

While Mr. Thomas does not go quite so far as to set up a demand-supply analysis, such an analysis is implied in his Table 11. That table would have been more helpful if he had included an estimate of consumptive losses (evapotranspiration) which are far more important than the amount of water withdrawn for use. One other way in which our resources of water are depleted is through excessive degradation in quality. Such degradation restricts reuse. It seems important, therefore, to include in a water-supply study of this nature some estimate of the net effects resulting from the use of water.

The writer is convinced the United States must soon recognize the need for a nationwide accounting for water and its disposition. The evidence that we must manage our water resources more intelligently is overwhelming, yet we have done very little as yet to view the problem in broad perspective. The proposal for a continuing national inventory of water as outlined in House Document 706,(1) has not yet been put into operation even though the concept is sound and the need pressing.

Estimation of future water requirements is indeed a very speculative business as the writer can testify from personal experience. The author has attempted to estimate requirements by the time our population has reached 450 million, and surely this is speculation in its extreme form. This has been done largely on the basis of extrapolation. The writer would like to point out some of the pitfalls of such long-range speculation.

We commonly accept the statement that water is an essential factor in all our economic pursuits and as a corollary, we assume that the current unit usage of water is necessary to our way of life. This blind acceptance of unit requirements as a mandatory requirement is not completely realistic. Except for use of water to sustain life, both animal and plant forms, we can and will use substitutes for many of the current usages of water. We can also greatly reduce unit uses for many purposes. Even in the use of water for irrigation, we are very generous. Our current irrigation practices do not

a. Proc. Paper 1754, September, 1958, by Robert O. Thomas.
1. Staff Engr., Director's Office, U. S. Geological Survey, Washington, D. C.

derive high efficiencies in the use of water, and in the future we may well look back upon the present irrigation practices as primitive and profligate.

Use of water for most industrial purposes is, in part, a convenience. This is especially true of the quantities being used. The opportunities for reducing use of water in industry are tremendous. Water for cooling purposes, while currently a common practice, is by no means a mandatory requirement. Air cooling and recirculation of water provide alternatives which are already in use to some degree.(2)

An analysis of current uses of water for industrial purposes reveals a wide range in practice. For example, use of water in steel production ranges from near minimal quantities used at the Fontana Plant of Kaiser Steel Company to rather large uses in the humid east. Unit usage ranges in a ratio of roughly 1 to 50. However, in the case of limited usage practically 100% of the water is evaporated. Similar ranges in practice in use of water for other industries occur.

The author's analysis of future use of water for agriculture, mainly irrigation, requires careful study. In Fig. 6 of Proc. Paper 1750 by Keith H. Beauchamp there is evidence that the author's estimate of 1.25 acre feet per acre required for irrigation in the humid portions of the United States is excessive. Furthermore, the total moisture deficiency shown by Beauchamp need not be met for successful crop production. Thus, Mr. Thomas' estimate of 124 million acre feet per year for future water requirements for the humid portion of the United States seems much too high.

The author seems to imply that irrigation agriculture in the semi-arid zone of the United States is inherently preferable to growing of crops elsewhere. This follows from the data in Table III from which he concludes that arid-zone irrigation is inherently more productive because it produces crops of approximately double the value per acre as compared with other types of agriculture. He seems to miss the point that irrigation agriculture is intensive regardless of its location while much of general agriculture is extensive in character.

In projecting irrigation requirements for semi-arid portions of the United States, he concludes that an ultimate water requirement of 3.25 acre feet per acre for some 147 million acres or a total requirement of 478 million acre feet. Two factors require further scrutiny here. The assumed requirement of 3.25 acre feet per acre seems much too high. Much of the semi-arid zone not now irrigated is currently growing dry-land crops, especially grains, and has an annual precipitation, concentrated during the growing season, of 12 inches to 20 inches. For example, in the Missouri Basin, according to Thornthwaite, average water deficiency ranges from 3 inches to 20 inches.(3)

The other factor requiring scrutiny is the source of supply for the computed requirement of 478 million acre feet of water. The only area of any substantial water surplus, in the order of magnitude required to satisfy the estimated demand, is the narrow strip on the Pacific Northwest Coast. This area of surplus is many hundreds of miles from the points of possible use. Furthermore, the other demands for that water in the same general area will be competitive with its use for irrigation.

One further feature of the agricultural requirements analysis is the author's assumption that the land available for agriculture is fixed. Many millions of acres of agricultural land in humid portions of the United States can be developed from swamp lands and from lands now used for forests or other low value uses.(4) Furthermore, such lands can even now be developed

for costs far less than the cost of land in many proposed irrigation developments.

While the author does recognize increasing productivity per acre, he discounts it as a major factor in future food production. He seems, also, to overlook the many other possible ways in which food might be produced, such as greater use of the sea environment. In the future, much of the extensive farming in the humid areas will be replaced with intensive farming, thus productivity per acre will increase through this means alone.(5)

The author has done a service in pointing out the nature and scope of our national water problem and the suggestions of methods by which the problems can be resolved are timely and constructive. We must do many things and invest much capital to insure future supply of water. The writer is confident, however, that the people of the United States have the resources and resourcefulness to solve their water problems. While we do not yet have all the answers to tomorrow's problems, we have faith that our system of government and our intellectual capacities will find the answers. Our history fully justifies such confidence.

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5. Heisig, Carl P.; Wooten, Hugh H.; and Christensen, Raymond P., "Balanced Development of Resources" in "Land," the U. S. Department of Agriculture Yearbook, 1958, pp. 584-592.

ALFRED R. GOLZE,[1] M. ASCE.—Mr. Thomas' paper, while it presents a number of opportunities to challenge and to rearrange his statistics, taken as a whole, is a conservative argument for civil engineers, in particular, and public works administrators, in general, to appreciate the seriousness of water conservation requirements throughout the United States and the need for appraisal of those requirements on a long-range basis.

Before discussing Mr. Thomas' summary, one or two of his internal points may be worthy of comment. In his introduction, he states that: "This paper does not evaluate the effect of varying monthly flow in the streams of the country, but presumes that sufficient storage, either above or below the surface of the ground, will be provided to regulate that portion of the available supply which is determined to be usable, with consideration to all competing demands." This is rather a strong assumption in that conservation storage

1. Asst. Commissioner, Bureau of Reclamation, U. S. Dept. of the Interior, Washington, D. C.

of the major rivers in the East such as the Connecticut, the Hudson, and the Susquehanna, to mention only a few, is unlikely to occur in the degree necessary to meet requirements and fresh water will flow unused to the ocean indefinitely, unless the engineers of the future devise some other way to secure the conservation benefit of river flows without storage.

Mr. Thomas also assumes here that necessary diversion and transmission of water supplies will be accomplished to the "maximum extent practicable." Apparently, the author means to the maximum "economic" extent practicable. Then, this too becomes a rather strong assumption. If the country could ever afford to pay the price for a gallon of water delivered at point of consumption that it now pays per gallon for crude petroleum delivered at the refinery, then we have no problem, but that time is a long ways off—if ever.

In Table 7, Mr. Thomas has estimated agricultural water requirements in the United States. In this table he shows irrigated acreages in millions of acres for certain populations. For a 200 million population he indicates that the semi-arid region is estimated to have 32 million irrigated acres, which is not an unreasonable prediction. However, for a 250 million population, which in point of time would not be long after the 200 million population, and which is an increase of 25%; the table shows 79 million irrigated acres, an increase of 147%. It would be practically impossible to increase our irrigated acreage in the semi-arid region by the amount indicated in this table, from the viewpoint of both available water and land, to say nothing of cost. The final figure in Mr. Thomas' Table 7, where he shows 147 million irrigated acres in the semi-arid region for a population of 450 million, is in excess of published figures on the ultimate development of the Western States. It would be interesting to see the 120 million new acres plotted on a map with a name source of water for each one. As far as the work of the Bureau of Reclamation is concerned, its long-range water studies do not, as yet, show any proposals leading to acreage of that magnitude under the most ideal economic conditions.

As stated before, the writer has no quarrel with the author of the premise at hand. We are all concerned over the growing seriousness of fresh water shortages now spreading throughout the United States. His paper, as he indicated in his introduction, omits a discussion of the number of elements which has some bearing on conservation of water. For example, Mr. Thomas could have mentioned that inventions presently unknown but certain to be developed in the next century will have a large impact on the consumption of fresh water, just as air conditioning has had in the past two decades. Also, with research getting more attention in such fields as evaporation control, we may expect some counter balance to consumptive use.

Of the ten items listed in the summary section, at the end of Mr. Thomas' paper, we have no serious question with the first five but the second five justify additional comment in the interest of clarifying our understanding of what the author evidently has in mind.

He identifies Item No. 6 as "accelerated research in the process of demineralization of saline or brackish water." The Department of Interior has had under way since 1950 an extensive program of research into the various processes for economic conversion of saline or brackish waters into fresh water. Congress at its last session authorized the construction of five pilot plants and when funds are made available for their construction this phase of the work will go ahead. It is to be expected that within the next ten-year period this phase of fresh water development may be an important step forward to the

point that use of saline or brackish water resource for industrial or municipal use may be a reality in areas where underground or precipitation flows are currently dwindling or high development costs prevail.

Mr. Thomas' Item No. 7 calls for construction and operation of sea-water conversion plants along the coasts. It could be supplemented by pointing out that the Department of the Interior's Saline Water program also provides for conversion of brackish waters at inland sources and that important bodies of inland waters that are too saline for industrial, municipal, or irrigation uses are being studied for possible treatment and conversion to fresh waters.

In Item No. 8, Mr. Thomas suggests that nuclear electrical generating power should be located near the sea coast with the output transmitted considerable distances inland. It has been our understanding that the apparent great economic value of nuclear generating plants is their freedom from fuel source making it possible to place them close to load centers without reference to source of fuel. Locating nuclear plants along the sea coast to secure the benefit of sea water for operation would place a greater economic value on water than on the cost of output transmission. With the large deposits of coal still to be used and petroleum available at reasonable cost, sea-side nuclear power plants do not appear to be part of the water conservation picture.

In his Item No. 9, Mr. Thomas refers to integrating the economies of the United States and Canada to utilize the combined land and water resources to the best interest of all. This work has been going on for some time through the International Boundary Commission, not only for Canada but also for Mexico. The adjustment of the water flows of international streams entering or leaving Canada and Mexico involves complicated international legal and administrative, as well as engineering, matters. As they are brought to a conclusion, the full use and development of the international stream flows may be expected.

In his Item No. 10, Mr. Thomas suggests that all industry in the United States should be relocated to the sea coast with agriculture taking up the interior of the country. He recognizes that an ideal world-wide peace would have to exist to make this possible. Until that situation occurs, Mr. Thomas' No. 10 item will have to be filed for future reference. It might be noted in passing, that as far as California is concerned, industry seems to be making a beaten path to the sea coast, and California water problems undoubtedly stem in large measure from that fact. Whether under ideal peace conditions it would be a good idea to have all industry strung up and down our coast lines (with spectacularly costly housing and traffic problems) and the interior of the country devoted to agriculture would be an interesting matter for our economists to debate. I suspect that the unbalanced geographical economy would not be well received by present generations. It would require a long period of educational preparation if such a wholesale reshuffling of our economy ever became necessary.

If Mr. Thomas' paper is read by one outside the field of water resource he might gather that nothing at all is being done today to protect the average American against a dry water faucet tomorrow. Water is still about our cheapest commodity for services rendered and today most of the country has plenty of fresh water for its miscellaneous needs, reflecting the foresight of the engineers and planners of yesteryear. But, as Mr. Thomas well knows, in his own state, they are working as best as they can to proceed with the California State Water Plan, but the State is presently of two minds as to the

best way it should be carried forward. Undoubtedly, a single effort will eventually come forward and the State will move ahead as the pressure of public opinion grows. The State of Texas, which alternately suffers from drought and extreme floods, is an ideal region for water conservation. It also has a State Water Plan, based on assistance from several Federal agencies, which is being readied for Congressional authorizations. This State can be expected to work out detailed solutions to its problems before extreme crisis arises. Other states in the West are either ahead of their problems or are working to meet them as they come their way.

In the East where the states are smaller and the urban developments well entrenched, it is more difficult to develop a coordinated plan, but the big cities like New York have worked out a solution and are keeping abreast of their requirements. The Nation's Capital, which is furnished its water supply by the Army's Corps of Engineers, is agitating for storage on the Potomac River or its tributaries to provide a long-range supply for the years ahead.

These are but examples. Mr. Thomas has called attention to a growing problem that we as civil engineers in the irrigation and water conservation field of activity cannot afford to let rest. The challenge has been well met to date, we are hard at work on tomorrow's supply, but it is the supply for the day after tomorrow, that we must keep ever in mind.

DONALD McCORD BAKER,[1] M. ASCE.—The author has assembled some very interesting facts regarding availability and use of water in this country, and from them reached a series of conclusions, also interesting, with some of which, however, the writer is not fully in accord.

Annual Runoff

In Table II, the author gives the average annual runoff for the 11 drainage basins of the United States as 1,318,000,000 acre feet, or 436.4 acre feet per square mile for the 3,020,000 square miles of this country.

Table No. A of this discussion expands the author's Table No. I, to show the average annual runoff per square mile for each of the above 11 basins. This quantity ranged from a maximum of 1027.0 acre feet per square mile for the North Atlantic Basin, or 235.3 per cent of the average, down to 58.1 acre feet per square mile for the Great Basin, or 13.3 per cent of the average. The average annual runoff from the North Atlantic Basin is 17.7 times that of the Great Basin.

No data is given by the author as to maximum or minimum annual runoff from these basins, nor as to cyclic variations in runoff.

Available Water Supply

The heaviest average annual runoff per square mile naturally occurs in those basins having the heaviest precipitation. Average slope of these basins, soil cover, and evaporation and transpiration losses also affect the average annual runoff.

Not all of this runoff, however, can be made available for use, even in those basins where average annual fluctuations in runoff are small, and low cost storage is ample and adequate.

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1. Cons. Engr., Los Angeles, Calif.

TABLE NO. A (a)

ANNUAL RUNOFF IN THE UNITED STATES - 1921-1945

Basin	Area Square Miles	Average Annual Runoff Total	Acre Feet per Square Mile
North Atlantic Slope	148,000	152,000,000	1,027.0
South Atlantic Slope and Eastern Gulf of Mexico	284,000	235,000,000	827.5
Mississippi River	1,250,000	449,000,000	359.2
Hudson Bay	48,000	3,600,000	75.0
St. Lawrence River	130,000	101,000,000	776.9
Western Gulf of Mexico	320,000	39,800,000	124.4
Colorado River	246,000	16,700,000	67.9
Great Basin	215,000	12,500,000	58.1
Pacific Slope in California	117,000	57,900,000	494.9
Columbia River and Coastal Streams in Oregon and Washington	262,000	250,000,000	954.2
Totals	3,020,000	1,317,500,000 (b)	436.3 (b)

(a) Note to Editor: Please give this Table its proper number in Paper and Discussion

(b) In Table I, as printed in Paper, Total Average Annual Runoff in acre feet is given as 1,318,000,000, and Runoff per square mile, on this basis, would be 436.4. The figures 1,318,000,000 and 436.4 respectively are used in this discussion.

Runoff to Supply Navigation

In many sections of the country, much runoff is required to supply water for navigation. Were this water not available for this use, large and costly railroads and highways, in addition to those now constructed, would be required as a substitute means of transportation for moving goods and people.

Runoff to Supply Domestic Use

In suburban and rural areas, the daily per capita use of water for human beings is probably around 50-60 gallons. Use for stock in certain rural areas may double this.

Use for Irrigation

The author, based upon records of value of crops per acre in the years 1939-1944 and 1949, from about 3 to 5 million acres irrigated on projects of the U. S. Bureau of Reclamation, and some 324 to 352 hundred million acres in cropped acreage in the entire United States during the same period, arrives at the conclusion that the value of crops from irrigated acreage in this country is twice that of crops from unirrigated harvested acreage, and later on in his paper states that by the time national population reaches 450,000,000 persons (see Table II, which estimates that this will occur in the year 2070) there will be 602,000,000 acre feet of water used annually for irrigation.

In reaching the above conclusion, general figures are used. It must be borne in mind, first, that most of the irrigable but unirrigated acreage occurs in the arid sections of the country—west of the 100th Meridian—also, that land included in U. S. Bureau of Reclamation projects is of a better type than cropped land in general, second, that in all probability, there is not sufficient water in the drainage basins located west of the 100th Meridian to irrigate all unirrigated land in the area, and water would have to be imported to the area from the humid region, provided that such importation was feasible. This is a large topic and the author does not discuss it.

Two of the most important questions which arises in connection with the above are—

1) How much land exists—in acres—and where is it located, in the arid section of the country, and how much water would such land require for irrigation.
2) Would the cost of bringing such land under irrigation be within the limits of feasibility.

Amount of Water Consumed in Various Uses

In reviewing the various data presented by the author, it appears to the writer that quantities referred to by him are those of water used, not of water consumed. If this is the case, a certain portion of the water used will ultimately return to stream flow, through ground water basins, sewage treatment plants, etc., and much of it will probably be available for reuse. No data is presented from which net consumptive use of water used may be estimated.

Use of Water for Steam Power Generation

The author presents figures of water consumed in producing steam electric power, and assumes (Table II) that when the national population has reached 450,000,000, 90 per cent of the power produced will be from steam, and 10 per cent from hydro. No data is given to support these figures, but it seems to the writer that, with all of the domestic, irrigation and navigation development which will be necessary new hydro-electric power sources will be brought into being to increase this hydro percentage.

In Table II the author estimates that by the year 2070, or about 115 years from now, the national population will have reached 450,000,000, and the population and use of water by that date will be the following, times the 1955 use—

Population	2.72
Domestic and Industrial	5.2
Irrigation	6.6
Electric Power Generation	4.9
General Industry	5.0

The author also states (Table II) that this population of 450,000,000 will be using 1,215,000,000 acre feet per year, or 92.2 per cent of the total annual supply.

The last statement assumes that actual streamflow, as it occurs in a state of nature, can be modified—through storage, and brought to places of use, in a feasible manner—much of the water occurring in surplus areas of the humid sections of the country being developed and transported vast distances to the arid sections, at feasible costs.

It should be noted that by the year 2070, practically one-half of the water used by this estimated population of 450,000,000 will be for irrigation, and about one-quarter of it for electric power development. Power can, in general, be transmitted from place of generation to place of use at feasible cost. In many instances irrigation water cannot be.

CONCLUSIONS

The author presents 10 courses of action which, in his opinion, will tend to counteract or offset the adverse water supply situation described in his paper.

The most striking of these is the relocation of industry to the sea coast, reserving the interior sections of the country for agriculture to the maximum possible extent.

Such a policy, if carried out, would completely change the character of our national economy. It would mean the location of the bulk of our population on the sea coast, a complete change in our transportation system to carry agricultural and mineral products to industrial areas for processing, and return many of them inland to points of consumption. Some of his conclusions, pertaining to conservation, are well founded, however.

Comment

The author does not discuss the manner in which he reaches his estimated 450,000,000 national population by the year 2070. As a general thing, improvement in living standards of a people reduces both birth and death rates, and the writer seriously questions the author's estimate of the above national population being reached by the year 2070.

The author has based his conclusions almost solely upon generalities and averages—something it is not possible to do in a study of this nature. Economic feasibility will enter into many of his recommendations, such as bringing surplus water from basins of high yield to areas of low yield for use therein for irrigation, the use of treatment of saline water located at sea level, and transportation of such water inland, in many cases over mountain ranges.

The idea of a more widespread water policy of water conservation in this country is sound, but along with it must be carried out a detailed study of the feasibility of many of the things suggested.

There is little question but that as time goes on, this country will have to be giving some thought to development of such a policy. If the author's paper does little else than start people thinking along this line, it will have served its purpose.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(EM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 (HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 (ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 (SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c]

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858 (SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW1), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958 (CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968 (SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977 (WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

c. Discussion of several papers, grouped by divisions.

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VOLUME 85 PART 2

Your attention is invited

NEWS OF THE IRRIGATION AND DRAINAGE DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE IRRIGATION AND DRAINAGE DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
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NEWS

March, 1959

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* * *

Carl R. Wilder, Newsletter Editor
3223 So. Columbine St., Denver 10, Colo.

CALENDAR OF COMING MEETINGS

May 4-8, 1959. National Convention, ASCE, Hotel Cleveland, Cleveland, Ohio. Irrigation and Drainage Division plans to sponsor three half-day sessions; on Groundwater, Water Rights, and Research.

May 11-12, 1959. National Conference, U. S. National Committee of International Commission on Irrigation and Drainage, Riverside Hotel, Reno, Nevada.

July 1-3, 1959. Annual Conference, Hydraulics Division, ASCE, Colorado State University, Fort Collins, Colo.

August 28-29, 1959. (dates subject to change) Annual technical conference, Irrigation and Drainage Division, ASCE, Denver, Colo. Probable theme, "Weather Control".

Note: No. 1959-13 is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 85, IR 1, March, 1959.

October 19-23, 1959. National Convention, ASCE, Hotel Statler, Washington, D. C. Irrigation and Drainage Divisions plans to sponsor three half-day sessions.

MONTHLY SCIENTIFIC PUBLICATION OF INTEREST TO IRRIGATION AND DRAINAGE DIVISION MEMBERS

It will be of interest to our Division members to know that the American Geophysical Union is combining the scientific and technological material in its bimonthly TRANSACTIONS with the quarterly JOURNAL OF GEOPHYSICAL RESEARCH and will issue the JOURNAL OF GEOPHYSICAL RESEARCH as a monthly starting in 1959. This will contain much material bearing on Hydrology of interest to our members.

The subscription rate for the new JOURNAL is $16.00 for the calendar year. Memberships, however, are invited. Membership is $10.00 per calendar year and includes a subscription to the JOURNAL. Any who are interested should write to the office of the American Geophysical Union, 1515 Massachusetts Avenue, N. W., Washington 5, D. C.

COMMITTEE ON WATER CONSERVATION

After the announcement in the Division September Newsletter that a new technical "Committee on Water Conservation" was being organized to function through five Task Groups, Chairman Blaney received many requests from throughout the United States to be included in one of the Groups. Therefore, the Executive Committee of the Division at its November 1958 meeting in Los Angeles authorized the appointment of associate (or correspondent) members of the Committee on Water Conservation.

The purpose of the parent (or control) committee is "to study and report on problems connected with the conservation of water supplies for irrigation, domestic and insutrial use; to promote the compilation and collection of data pertaining to water conservation, giving particular attention to the following:

a) Methods of conserving water
b) Consumptive use of water
c) Re-use of drainage water and reclamation
d) Watershed management
e) Water quality

and to act as a clearing house for the coordination and dissemination of information on these subjects at meetings and Division Conferences."

Each member of the parent committee is Chairman of a Task Group. Members of the Committee on Water Conservation and Task Groups are as follows:

<u>Committee on Water Conservation</u>

Harry F. Blaney, Chairman

Art Bruington	Lloyd Myers, Jr.
P. H. McGauhey	Charles Thomas

Task Group on Methods of Conserving Water

Art Bruington, Chairman
William Balch, Gerald E. Carlat and Donald H. McKillop

Task Group on Consumptive Use of Water by Irrigated Crops and Native Vegetation

Harry F. Blaney, Chairman
Wayne D. Criddle, Clyde Houston and G. Marvin Litz

Task Group on Re-Use of Drainage Water and Water Reclamation

Lloyd Myers, Jr., Chairman

Keith Anderson	A. R. Robinson
Robert L. Lowry	Lyman S. Willardson

Task Group on Water Management

Charles Thomas, Chairman

Robert Edmonston	Myles R. Howlett
C. Warren Hink	Leonard Schiff

Task Group on Water Quality

P. H. McGauhey, Chairman

W. W. Aultman	Martin R. Huberty
Albert F. Bush	David B. Willets

Organization of the Committee was completed in December at a meeting in Los Angeles. A program for the Task Groups in 1959 was outlined and appointment of associate members approved. Most of the committee members are planning to attend the Society convention in Los Angeles in February and to meet the Division Executive Committee. The Committee is planning to sponsor a technical session at the National Convention in Reno June 19-23, 1960.

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Harry F. Blaney, Irrigation Engineer with the Department of Agriculture, recently was privileged to accompany five prominent Russian engineers on a tour of irrigated areas of the United States. The tour was sponsored by the Department of Agriculture in cooperation with the State Department and several of our state agricultural colleges. Visits were made to the state universities of Illinois, Nebraska, Colorado and Utah and Texas Technological College. At those colleges and universities visitors conferred with staff members working on irrigation, drainage and conservation problems. They also met with staff members of the Agricultural Research Service and Soil Conservation Service and enjoyed a number of field trips to visit farms and watersheds where they were able to see the results of irrigation, drainage, and soil and water management practices.

Some of the objectives of the tour were:

1. To become acquainted with farmers' irrigation and drainage problems and to learn how organizations and institutions assist them in solving these problems.

2. To observe water utilization, survey planning and scientific research concerned with irrigation development.
3. To visit United States Department of Agriculture research centers and Land-Grant Colleges for:
   (a) Consultation with officials, agricultural engineers and specialists in irrigation and drainage techniques.
   (b) Conferences with extension irrigation specialists regarding methods of disseminating information and providing advice in irrigation and water conservation practices.
   (c) To review education programs through which agricultural engineers are trained for field work.
   (d) To observe soil and water conservation at research stations and areas including measurements of soil and water losses under different systems of land management.
4. To visit Soil Conservation Service state offices and projects to observe soil conservation practices established for the use of irrigated agriculture and efficient water use on the farm.
5. To observe how low, wet land is reclaimed for farming.
6. To observe the irrigation of cotton and other crops through the application of the various irrigation techniques.
7. To observe the utilization of underground water and recharge.

Mr. Aleksandr Askochenskiy, leader of the Soviet delegation, commented at one of the interviews with newspaper reporters: "Such tours are mutually beneficial, both to us and Americans touring Russia. Such an exchange is beneficial in all fields and serves to strengthen the friendship between the United States and Russia".

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William W. Donnan, Secretary of the I & D Division Executive Committee, was a member of a team of agricultural scientists in the soil and water management field which recently was privileged to visit Russia on a cultural exchange mission. They had 33 days in the Soviet Union during which time they talked with scientists, administrators and farmers and observed research and its application to soil and water management problems. Following their visit to the USSR, Mr. Donnan also visited Denmark, The Netherlands and England to study soil and water management research in those countries.

With regard to irrigation in Russia, Donnan reports that the large scale developments seemed to be hydraulically well designed and engineered, while farm irrigation practices are for the most part quite primitive. The lands to be irrigated are not as carefully levelled as in the irrigated areas of the western United States. Water application seemed to require tremendous amounts of manpower and individual hand labor.

The USSR is well aware of its tremendous drainage problems and presently seems more concerned with those problems in humid areas than in irrigated areas. They are carrying on a program of research in humid area drainage. Their program of research dealing with salinity and alkali problems is not so good as that of the U. S. Salinity Laboratory. In the application of drainage techniques, the USSR is far behind the USA. They have some very deep open drains and trunk drainage systems but as yet very few tile drains have been installed.

Approximately 40 per cent of the arable land in Denmark is drained by tiles. At present most tile drains are 2 inches in diameter, but the trend is toward using 3-in. diameter tiles in the future. Tile drainage in Denmark started in 1850. The drainage of agricultural land is subsidized in part by the government. The main drainage systems and pumping stations are all constructed by the government, which also shares in the cost of individual farm drainage systems provided the plans are approved.

Mr. Donnan visited several institutes and laboratories in The Netherlands which are concerned with soil and water management problems. Some of the basic and applied research work presently being carried on in The Netherlands could be of considerable importance and assistance in the United States.

In England Donnan visited the University of Cambridge School of Agriculture. Research work being carried on in the laboratories at Cambridge and also in the field could be of considerable importance to drainage engineers of the U. S.

Mr. Donnan's printed notes on his visit to Russia and other European countries contain considerably more detail on the work being done and the names and addresses of responsible parties at each of the locations visited. Interested members of ASCE no doubt could obtain those details from him.

In December 1958, Section E—Geology and Geography—of the American Association for the Advancement of Science sponsored a symposium on water and agriculture. This symposium consisted of four half-day sessions dealing with the general subject of sources and uses of water in agriculture. The ten co-sponsors of the symposium included the American Society of Civil Engineers. John G. Sutton, M. ASCE, drainage engineer, Soil Conservation Service, represented the Irrigation and Drainage Division at this meeting.

Society members appearing on the program included:

Donald A. Williams, Administrator, Soil Conservation Service.
Carl B. Brown, Soil Conservation Service.
William C.Ackerman, Chief, State Water Survey, Urbana, Ill.
Waldo E. Smith, Executive Secretary, American Geophysical Union.

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The Hydraulics Division of the Colorado Section sponsored the January 1959 joint meeting of the Hydraulics and Irrigation and Drainage Divisions of that section. Thad McLaughlin, District Geologist, USGS, spoke on the subject, "Ground Water Resources of Colorado." He discussed the occurrence of ground water in Colorado with particular reference to four principal aquifers:

Alluvium in the Arkansas Valley
Alluvium in the South Platte Valley
Basin fill in the San Luis Valley
Ogallala formation in the high plains.

He also discussed the extent and effects of the development of irrigation from wells.

The February 1959 meeting of these divisions was under the sponsorship of the Irrigation and Drainage Division. John T. Phelan, Irrigation Engineer, SCS, Lincoln, Nebr., spoke on "Irrigation Efficiencies".

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The program for the first technical meeting of the U. S. National Committee of the International Commission on Irrigation and Drainage is being arranged by the Technical Activities Committee, C. W. Thomas, M. ASCE, Chairman. Probable authors of papers on the four major questions to be discussed are the following:

<u>Question 11. Reclamation of Waterlogged and Marshy Lands.</u>

Victor I. Myers and Ronald C. Reeve, Dean C. Muckel and Dr. Frank G. Viets, Ray J. Winger, Jr., A. M., ASCE, and Dr. Arthur F. Pillsbury.

<u>Question 12. Sprinkler Irrigation and Comparison with Other Methods of Irrigation.</u>

Prof. E. H. Kidder, Dr. Robert M. Hagan and Yoash Vaadia, Dr. Howard R. Haise and Lloyd E. Myers, A. M. ASCE, and Percy M. Pharr, Jr.

<u>Question 13. Tolerance of Plants to Minerals in Solution in Irrigation Water and Soil.</u>

Milton Fireman and Dr. Howard B. Peterson.

<u>Question 14. Use of Longitudinal Embankments or Levees as Flood Protection Measures.</u>

Harry W. Adams, M. ASCE, Robert C. Woodson, and Preston T. Bennett, M. ASCE.

This meeting will be at the Riverside Hotel, Reno, Nevada, May 11 and 12, 1959.